AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

THIRTEENTH ANNUAL CONVENTION

Held at Chicago, Ill.,

February 8, 9, 10, 1917

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AMERICAN CONCRETE INSTITUTE.

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	1916	WILLIAM P. ANDERSON. ERNEST ASHTON. EDWARD D. BOYER. CHARLES A. GOW. ALFRED E. LINDAU. JOHN G. TREANOR.

BY-LAWS.

Article I.

MEMBERS.

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Article II.

OFFICERS.

Section 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next

Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty, of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President at the first regular session of the annual convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

SEC. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

SEC. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

SEC. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

SEC. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

ARTICLE IV.

DUES.

SECTION 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

SEC. 2. The annual dues of each member shall be ten dollars ($10.00).

SEC. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amonut of dues for that fiscal year; but he shall not be entitled to a copy of the Proceedings of that year.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Institute.

Article V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

Section 1. Proposed Recommended Practice and Specifications to be submitted to the Institute must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter, such Recommended Practice and Specifications shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative.

Article VI.

AMENDMENT.

Section 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE THIRTEENTH ANNUAL CONVENTION.

Hotel La Salle, Chicago, Ill.

First Session, Monday, February 8, 1917, 10 a. m.

The Convention was called to order by the President, Leonard C. Wason.

The President announced the appointment of the following Committee on Resolutions:

William M. Kinney, *Chairman*, Chicago.
Arthur N. Johnson, Chicago.
Arthur J. Maynard, State Farm, Mass.
Charles R. Gow, Boston, Mass.
David J. Merran.

The President announced the appointment of a Special Committee to investigate the recent fire in a concrete building at Far Rockaway, N. Y., the personnel to be announced later.

The following papers were then read and discussed:

"Slag as Concrete Aggregate," by Sanford E. Thompson, Consulting Engineer, Boston, Mass.

"Cost Accounting and Organization," by Leslie H. Allen, Aberthaw Construction Co., Boston, Mass.

Discussion by Frank R. Walker, Chicago, Ill.

"Relation between Engineers and Contractors," by C. A. Crane, Secretary, The General Contractor's Association, New York City; read by F. C. Wight, in the absence of the author.

"Field Tests of Concrete for the New York Subways," by J. C. Steinle, Assistant Engineer Public Service Commission, New York City.

"Building Codes for Small Towns," by Ernest McCullough, Consulting Engineer, Chicago, Ill.

Second Session, Monday, February 8, 1917, 8 p. m.

President Leonard C. Wason in the chair.

The report of the Committee on Reinforced Concrete and Building Laws was presented by the Chairman, E. J. Moore. On motion the "Standard Building Regulations for the Use of Reinforced Concrete," presented by the

Committee was adopted by vote of 14 to 10, and submitted to letter ballot of the Institute for adoption as Standard. (By motion later in the Convention this action was rescinded. See record of Fourth and Fifth Session.)

The following papers were then read and discussed:

"Flow of Concrete," by E. B. Smith, U. S. Bureau of Standards, Washington, D. C.

"Recent Tendencies in Industrial Building Construction," by William P. Anderson, President, Ferro Concrete Construction Co., Cincinnati, Ohio.

"Extensometer Measurements in a Reinforced Concrete Building over a Period of One Year," by Arthur R. Lord, Consulting Engineer, Chicago, Ill.

"Tests on Thin Flat Dome of Concrete Tile," by Willis A. Slater and C. R. Clark, University of Illinois.

"Treatment of Concrete Ornamental Elevated Stations, Dual System of Rapid Transit," by S. J. Vickers, Architect, Public Service Commission, New York.

"Unit Construction," by John E. Conzelman, Civil Engineer, St. Louis, Mo.

THIRD SESSION, TUESDAY, FEBRUARY 9, 1917, 10 A. M.

President Leonard C. Wason in the chair.

The progress report of the Committee on Reinforced Concrete Standpipes was read by the Secretary.

The paper on "Concrete Piles, Plain and Reinforced," was read by Charles R. Gow, Boston, Mass., its author.

The report of the Committee on Sewers, Plain and Reinforced, was presented by its Chairman, W. W. Horner, of St. Louis, Mo. The proposed Standard Specification for Concrete Sewers was discussed and ordered printed.

The following papers were read and discussed:

"Sewer Problem Due to Subways in New York," by Samuel D. Bleich, Assistant Division Engineer, Public Service Commission, New York City; read by H. B. Alvord in the absence of the author.

"A Course of Instruction in Reinforced Concrete," by Prof. William K. Hatt, Purdue University.

A paper, "Effect of Hydrated Lime on the Strength, Absorption and Expansion of Concrete," by Prof. H. H. Scofield, Purdue University, was on the program but Prof. Scofield was unable to be present. His paper is printed in the Proceedings. In his absence the general subject of hydrated lime was formally discussed by P. H. Bates, U. S. Bureau of Standards and Bela Nagy, Hydrated Lime Bureau, both of Pittsburgh.

A paper, "Does Concrete Construction Reduce Vibration?" by Morton C. Tuttle, Secretary, Aberthaw Construction Company, Boston, Mass., was read by Frank C. Wight in the absence of the author.

On motion the Convention adopted to be sent to letter ballot, as Standard No. 1 replacing the present Standard No. 1, the standard Cement Specifications adopted by the American Society for Testing Materials to become effective January 1, 1917.

FOURTH SESSION, TUESDAY, FEBRUARY 9, 1917, 7.30 P. M.

President Leonard C. Wason in the chair.

A. N. Johnson, Chairman, presented the report of the Committee on Concrete Roads and Paving. The report was adopted and the following Specifications included in it sent to letter ballot for adoption as Standards.

Specifications for Concrete Roads, Streets and Alleys.

Standard No. 5. One-Course Concrete Highway.
Standard No. 17. One-Course Concrete Pavement.
Standard No. 18. Two-Course Concrete Pavement.
Standard No. 19. One-Course Concrete Alley Pavement.
Standard No. 20. Concrete Pavement Between Street-Car Tracks.

The following papers were read and discussed:

"Friction of Concrete Slabs on Different Supporting Materials," by A. T. Goldbeck, Engineer of Tests, Office of Public Roads and Rural Engineering, Washington, D. C.; read by H. D. Hynds in the absence of the author.

"Essentials for the Successful Construction of Concrete Highway," by William M. Acheson, Division Engineer, State Highway Department, Syracuse, N. Y. (Printed in *Proceedings*, Am. Con. Inst. Vol. 12.)

"Condition of the Wayne County Roads," by A. N. Johnson, Consulting Engineer, Chicago, Ill.

"Maintenance of Concrete Roads in Connecticut," by W. Leroy Ulrich, State Highway Department, Hartford, Conn.

"Some Recent Developments in the Construction of Concrete Roads," by William M. Kinney, Engineer, Promotion Bureau, Universal Portland Cement Co., Chicago, Ill.

On motion the Final Report of the Joint Committee on Concrete and Reinforced Concrete was received by the Convention by unanimous vote.

On motion the Report of the Committee on Reinforced Concrete and Building Laws, ordered at Second Session to be sent to letter ballot was also

ordered to be accompanied with all the data on which the decisions therein were based. Vote 17 to 9. (This action was rescinded by later action in Fifth Session.)

Motion that Final Report of Joint Committee on Concrete and Reinforced Concrete be sent to letter ballot of the Institute as Recommended Practice was ruled out of order by the Chair on the ground that this report had not been before the Institute the constitutionally required 30 days.

FIFTH SESSION, WEDNESDAY, FEBRUARY 10, 1917, 10 A. M.

President Leonard C. Wason in the chair.

Business Session.

The President reported that the letter ballot had resulted in election of the following officers for the ensuing year:

President: William K. Hatt.
Vice-President: Sanford E. Thompson.
Treasurer: Robert W. Lesley.
Directors: *Third District*—W. P. Anderson.
Fourth District—Ernest Ashton.
Fifth District—Ernest McCullough.

The Secretary read the Annual Report of the Board of Directors.

The Secretary read the Auditor's Report.

The Committee on Resolutions read the following report:

WHEREAS, The papers presented at the 1917 Convention of the American Concrete Institute have been of unusual merit and have brought out considerable discussion; be it

Resolved, That the authors of these papers be tendered the thanks of the Institute; and be it further

Resolved, That the President be hereby instructed to address a letter expressing these thanks to each author.

WHEREAS, The progress of the American Concrete Institute during the year 1916 had been unusually well maintained, the papers provided for the 1917 Convention have been of unusual interest and merit; and

WHEREAS, Each session has shown a large and interested attendance; be it

Resolved, That the thanks and commendation of the American Concrete Institute be extended to President L. C. Wason for his untiring efforts and to the other officers and directors for their able counsel and assistance.

WHEREAS, The American Association of Engineers has extended an invitation to the American Concrete Institute to cooperate in a banquet, and those in charge of this banquet have worked diligently and faithfully to make it a successful and enjoyable occasion; be it

Resolved, That the American Concrete Institute extends to the American Association of Engineers and its officers expressions of deep appreciation and thanks them for their cordial interest.

On motion the following resolution was adopted.

It has been called to the attention of the 1914–15 Committee on Reinforced Concrete and Building Laws that the report of the Committee on Column Tests, as published, stated that the Engineering Experiment Station of the University of Illinois cooperated to the extent of furnishing observers in conducting the tests. It is desired to put on record that the tests were made distinctly as cooperative work, and that the Engineering Department of the University of Illinois was a participant in this investigation.

The following paper was read and discussed:

"Artistic Stucco," by John B. Orr, Miami, Fla.

On motion of E. J. Moore, Chairman of the Commitee on Reinforced Concrete and Building Laws, the report of that committee was ordered to go over for another year without action.

The report of the Committee on Building Block and Cement Products was presented by the Chairman, Robert F. Havlik. On motion the following Specifications were ordered to letter ballot for adoption as Standards:

Standard No. 21. Specification for Reinforced-Concrete Fence Posts.

Standard No. 10. Standard Specifications and Building Regulations for the Manufacture and Use of Concrete Architectural Stone, Building Blocks, and Brick.

On motion of the Chairman the Standard Specifications on Drain Tile of the American Society for Testing Material was adopted to be sent to letter ballot as Standard No. 9 of the American Concrete Institute.

On motion the following resolution was adopted:

That the Committee on Publications be empowered to so edit all communications intended for publication that they will conform to the Standards of the Institute, such editing to control the form but not the substance of such communications.

The following papers were then read and discussed:

"Ornamental Products," by A. G. Higgins, Manager, Trusswall Manufacturing Co., Kansas City, Mo.

"Cast Concrete in the Improvement of the Lake Front at Chicago," by L. G. White, Chief Engineer, South Park Commissioners, Chicago.

"Concrete Roofing Tile," by A. B. Tamm, Hawthorne Cement Products Co., Cicero, Ill.; read by Mr. Ferguson in the absence of the author.

"Concrete Silo Staves," by S. D. Playford, Elgin, Ill.

SIXTH SESSION, FEBRUARY 10, 1917, 3 P. M.

Mr. W. M. Kinney in the chair.

The following paper was read and discussed:

"Effect of Width of Slab on Effective Width for Design," by A. T. Goldbeck, Engineer of Tests, U. S. Office of Public Roads and Rural Engineering, Washington, D. C.; read by W. A. Slater in the absence of the author.

The report of the Committee on Nomenclature was presented by Leslie H. Allen of the committee.

The report of the Committee on Reinforced-Concrete Highway Bridges and Culverts was presented by W. K. Hatt of the committee. The report was received as information and ordered printed.

The report of the Committee on Concrete Aggregates was presented by Cloyd M. Chapman of the committee.

The report of the Committee on Reinforced-Concrete Chimneys was presented by J. E. Freeman of the committee.

President Leonard C. Wason took the chair.

The report of the Committee on Sidewalks and Floors was presented by J. E. Freeman, Chairman.

The report of the Committee on Fireproofing was read by the President.

President-elect William K. Hatt took the chair.

The President-elect declared the Convention adjourned *sine die*.

Papers Read Before the 13th Annual Convention of the American Concrete Institute

TESTS OF CONCRETE COLUMNS WITH CAST-IRON CORE.

By L. J. Mensch.*

Reinforced-concrete columns have hardly been in use 25 years. As long as reinforced-concrete buildings remained only a few stories high, the size of the columns did not become too large. Where buildings were exceptionally high, steel columns were regularly adopted.

About 15 years ago, M. Considère showed that spiral reinforcement allowed the use of much smaller columns, which were even cheaper and safer than columns with vertical reinforcement only.

The great economy of spirally hooped concrete columns over the steel columns and of reinforced concrete in general over steel construction, effected a nearly universal adoption of reinforced concrete for heavy and high buildings, but there the large size of the hooped columns often was found to be objectionable.

In a ten-story warehouse nearing completion in the city of Chicago, the difference in cost of steel columns and hooped concrete columns amounted to $120,000 and, of course, the concrete columns were adopted, although their diameters varied from 54 in. in the basement to 22 in. in the top story, and occupied an excess space over steel columns which was worth nearly $20,000.

The invention of hooped cast-iron columns by the well-known concrete pioneer, Dr. F. von Emperger, will allow in the future more reasonable sizes of columns in tall and heavy buildings. Dr. Emperger tested, about eight years ago, steel columns stiffened with concrete. He found that the concrete actually stiffened the steel columns so that they failed when the elastic limit of the steel was reached (which the particular steel columns tested alone did not reach) and besides the concrete contributed an additional strength of 1200 lb. per sq. in. of the concrete section. He noticed that the flow of the steel members after the elastic limit had been reached destroyed the concrete and placed a limit on the strength of the concrete at a unit deformation of about 0.001 in. per in.

Then Dr. Emperger conceived the idea that cast-iron with its much greater compressive strength, reinforced with hooped concrete, would make a more suitable column. Cast-iron with its low modulus of elasticity attains its maximum strength at a compression of about 0.02 in. per in. length and has a decided flow of material only near the ultimate strength.

It is of great importance to note that plain concrete reinforced with longitudinal steel rods fail at a unit compression rarely exceeding 0.0015 in. per in. length, and is, therefore, not suitable to help cast-iron with its low modulus of elasticity.

Hooped concrete, however, can stand much greater deformations without failure, and the combination of hooped concrete and cast-iron makes it possible

* Contracting Engineer, Chicago, Ill.

to develop the ultimate strength of both the hooped concrete and the high compressive strength of the cast-iron.

Fig. 1 shows the strain-stress curves for plain and hooped concrete and for various grades of cast-iron; and for mild steel. The strain-stress curve for

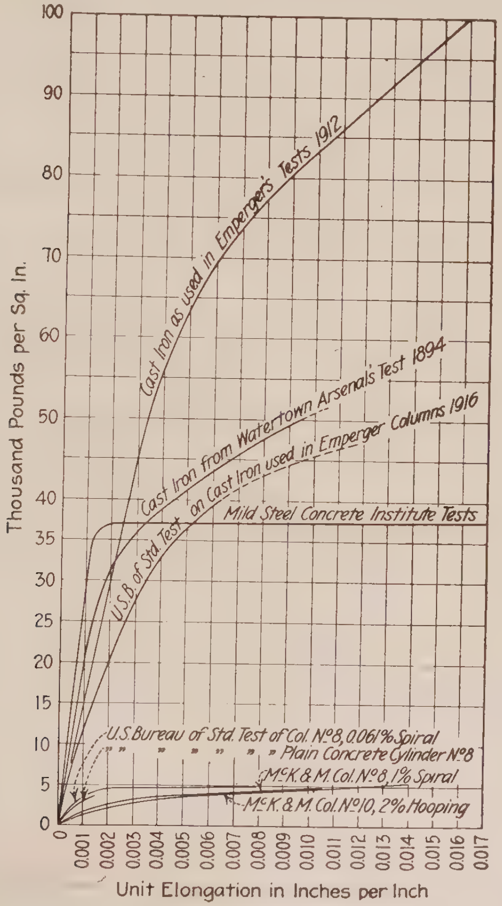

FIG. 1.—STRESS DEFORMATION CURVES ON COLUMN REINFORCING METAL.

plain concrete ends at a unit deformation of 0.0015 in. per in. length, at which deformation cast-iron is stressed only about 15,000 to 25,000 lb. per sq. in. The curves for spirally hooped concrete end at much larger unit deformations, and the larger the spiral reinforcing the larger is the unit deformation which the concrete can maintain without failure. At a unit deformation of 0.01 in.

FIG. 2.—SOME OF THE CAST-IRON CORED CONCRETE COLUMNS AFTER FAILURE.

FIG. 3.—SOME OF THE CAST-IRON CORED CONCRETE COLUMNS AFTER FAILURE.

per in. length, for example, cast-iron of ordinary quality will develop stresses from 45,000 to 50,000 lb. per sq. in., while high grade cast-iron such as Dr. Emperger used (and which according to Johnson's "Materials of Construction," any good foundry can produce) will develop stresses of over 80,000 lb. per sq. in., providing the column does not fail in bending. Fig. 1 further

FIG. 4.—CAST-IRON CORES TO CONCRETE COLUMNS AFTER THE COLUMNS HAD BEEN TESTED.

shows that mild steel cannot develop much more than 37,000 lb. per sq. in. in the most favorable case, in combination with concrete, with or without spiral hooping.

To verify these theoretical deductions, the writer had made at the end of September and the beginning of October, 1916, 20 columns from 6 ft. to 14 ft. long as shown in Fig. 5 and the accompanying table. The concrete was of

1 : 1 : 2 mixture, $12\frac{3}{4}$ in. outside diameter, 12 in. core diameter of spiral, reinforced vertically with 0.66 to 1.22 sq. in., and spirally with No. 8 gage wire with $1\frac{1}{8}$ in. pitch amounting to 0.61 per cent of the core area. The cast-iron reinforcing consisted in most cases of 6-in. cast-iron columns $\frac{3}{4}$ in. thick, widened at the ends. The length of the columns follow:

2 columns—10 ft. long
4 " 8 " "
8 " 10 " "
2 " 12 " "
2 " 14 " "
2 plain cast-iron columns 8 ft. long
3 " " " " 2 " "

With each batch one concrete cylinder 6 x 12 in. was made.

MATERIALS AND REINFORCEMENT.

The cement used was Universal Portland Cement, received from the warehouse of the Wisconsin Lime & Cement Co.; sand and gravel were the standard materials used in the city of Chicago; good clean washed torpedo sand, and gravel passing a 1-in. ring containing a mixture of materials from $\frac{3}{8}$ in. up.

The spiral reinforcement was furnished by the American System of Reinforcing, and was standard high carbon No. 8 wire fastened together at two opposite sides with $\frac{1}{4}$ in. round rods. The $\frac{3}{8}$-in. square deformed bars of high carbon steel were wired to the spirals in distances of about one foot. No tests were made on the steel reinforcement as the test specimens were lost in shipment.

CAST-IRON COLUMN CORES.

The columns were made by the American Building Foundry Co., of Chicago, of a mixture of 50 per cent of gray iron and 50 per cent of scrap. Test cubes of 1-in. sides tested by Prof. Duff A. Abrams, of the Lewis Institute, Chicago, showed a stress of 100,000 to 150,000 lb. per sq. in., but the actual castings, as typified by a 5-in. column 24 in. long, showed only an ultimate strength of 72,700 lb. per sq. in., and an initial modulus of elasticity of 10,000,000 lb. The casting varied only about $\frac{1}{8}$ to $\frac{3}{16}$ in. in thickness where the core was supported by chaplets, and between these points, varied as much as $\frac{1}{2}$ in. in thickness. The column ends were machined at the foundry. The sections were often 10 per cent less in area than the theoretical section.

MAKING OF THE CONCRETE COLUMNS.

The columns were made alongside a brick wall at the American Building Foundry Company's yard, in a vertical position. The ground for each column was well tamped and an iron plate about two feet square was tamped in the ground and fairly well leveled, and on top of this plate a wooden plate

	1	2	3	4	5	6	7	8	9	10
DATE OF TEST	December 1916 6th	7th	5th	5th	7th	6th	November 1916 28th	28th	December 1916 5th	6th
FIRST CRACK, LB.	436,500	-	436,500	570,000	-	565,500	490,000	440,700	-	-
MAX. LOAD, LBS.	1,057,000	1,026,000	940,000	1,031,500	975,000	920,000	518,500	562,200	480,000	540,000
LOAD ON CONCRETE	85□"@4780 Lb. = 407,000 Lb.		85□"@4780 Lb - 407,000 Lb		85□"@4780 Lb - 407,000 Lb					
LOAD ON CAST IRON	124□"@51,100" = 634,500"		124□"@43,500" = 540,750"		124□"@43,500" - 539,250"					
TEST CYLINDERS	4,300□"	4,300□"	3,750□"	3,750□"	4,150□"	4,150□"	3,630□"			
DATE OF CONCRETING	October 1916 2nd		September 1916 29th		30th		28th			
MARK OF CONCRETE CYLINDER 6"x12"	(1,2)		(3,4)		(5,6)		(7)	(8)		
THICKNESS OF WALLS OF CAST IRON MEASURED OPPOSITE SIDES AT TEST HOLES	3/4" & 11/16"	3/4" & 5/8"	3/4" & 11/16"	3/4" & 11/16"						
COMPRESSION AT ULTIMATE LOAD / LENGTH OF COLUMN	1/120		1/87	1/110					1/140	1/192

	11	12	13	14	15	16	17	18	19	20
DATE OF TEST	November 1916 27th	27th	29th	Dec 1916 1st	November 1916 28th	29th	December 1916 4th	4th	1st	2nd
FIRST CRACK, LB.	643,500	650,000	776,000	727,500	850,000	576,300	732,000	630,500	630,500	679,000
MAX. LOAD, LBS.	[illegible]	940,000	896,000	1,066,500	1071,800	996,000	991,500	909,000	888,500	827,500
LOAD ON CONCRETE	85□"@4780 Lb 407,000 Lb				100□"@4780 Lb - 478,000 Lb		85□"@4780 Lb - 407,000 Lb		85□"@4780 Lb × 95/100 = 387,000 Lb	
LOAD ON CAST IRON	124□"@44,100" - 546,375"				134□"@41,500" - 554,900"		124□"@42,100" = 523,250"		124□"@37,900" = 469,500"	
TEST CYLINDERS	3750□"	3750□"	4250□"	4250□"	4250□"	4250□"	-	-	4250□"	4,400□"
DATE OF CONCRETING	September 1916 25th		October 1916 2nd		September 1916 29th		28th		25th	
MARK OF CONCRETE CYLINDER 6"x12"	(11,12)		(13,14)		(15,16)		(17)	(18)	(19)	(20)
THICKNESS OF WALLS OF CAST IRON MEASURED OPPOSITE SIDES AT TEST HOLES	3/4" & 7/8"	3/4" & 7/8"					3/4" & 11/16"	3/4" & 11/16"	7/8" & 11/16"	3/4" & 11/16"
COMPRESSION AT ULTIMATE LOAD / LENGTH OF COLUMN							1/144	1/172	1/100	1/168

FIG. 5.—DETAILS OF THE CAST-IRON CORED CONCRETE COLUMNS TESTED.

Column No.	Outside Diameter, in.	Core Diameter, in.	Actual Area of Concrete within Core, sq. in.	Vertical Reinforcement. Square Bars.	Vertical Reinforcement. Area, sq. in.	Spiral Reinforcement. Diameter and Pitch.	Spiral Reinforcement. Equivalent Area.	Spiral Reinforcement. Percentage of Full Concrete Area.	Cast Iron. Outside Diameter, in.	Cast Iron. Thickness, in.	Cast Iron. Area, sq. in.	Cast Iron. Percentage of Actual Concrete Area.	Length of Column, ft.	Load at First Crack, lb.	Maximum Load, lb.	Average Maximum Load, lb.	Load Considered Carried By Concrete at 4778 lb. per sq. in.	Load Considered Carried By the Cast Iron Reinforcement. Total lb.	Load Considered Carried By the Cast Iron Reinforcement. lb. per sq. in.	Average Load per sq. in. Considered Carried by Cast Iron.	Vertical Unit Deformation at ½ Maximum Load.	Vertical Unit Deformation at Maximum Load.	Strength in lb. per sq. in. of 6 x 12-in. Cylinder.
1	$12\frac{3}{4}$	12	84.8	$4-\frac{3}{8}$ $2-\frac{1}{4}$	.66	8 gage $1\frac{1}{8}$ c.c.	.69	.61	6	$\frac{3}{4}$	(11.1)* 12.37	14.6	6	436 500	1 057 000						.000417	.00833	4300
2	"	"	"	"	"	"	"	"	"	"	"	"	6	776 000	1 026 000	1 041 500	405 000	636 500	51 400	(56 900)† 51 400	.0004	.0090	4300
3	"	"	"	"	"	"	"	"	"	"	"	"	8	436 500	940 000		405 000	535 000	43 200		.00034	.0115	3750
4	"	"	100.7	"	"	"	"	"	"	"	"	"	8	570 000	1 031 500		481 150	550 350	44 500		.000517	.0080	3750
5	"	"	93.5	"	"	"	"	"	5	1	12.57	13.4	8	679 000	925 000		446 600	478 400	38 100		.00035	.0074	4150
6	"	"	74.6	"	"	"	"	"	7	"	12.52	16.8	8	565 500	920 000		356 300	563 700	45 000	(47 500)† 42 700	.000366		"
7	"	"	113.1	"	"	"	"	"	..	..	....	....	10	490 000	518 500				....		.00025		3890
8	"	"	113.1	"	"	"	"	"	..	..	....	....	10	440 000	562 200	540 350	540 350	0	....		.000225		3370
9	6	$4\frac{1}{2}$	....	....	..	....	..	..	6	$\frac{3}{4}$	12.37		8		480 000								
10	"	"	....	....	..	....	..	..	6	$\frac{3}{4}$	"		8		540 000	510 000	0	510 000	(45 500)† 41 200				
11	$12\frac{3}{4}$	12	84.8	$4-\frac{3}{8}$ $2-\frac{1}{4}$	.66	8 gage $1\frac{1}{8}$ c.c.	.69	.61	6	$\frac{3}{4}$	12.37	14.6	10	643 500	911 000						.00035		3750
12	"	"	"	"	"	"	"	"	"	"	"	"	10	650 000	940 000						.00035		"
13	"	"	"	$8-\frac{3}{8}$ $2-\frac{1}{4}$	1.22	"	"	"	"	"	"	"	10	776 000	896 000						.00034		4250
14	"	"	"	"	"	"	"	"	"	"	"	"	10	727 500	1 066 500	941 375	405 000	536 375	43 400		.00045		"
15	"	"	99.7	"	"	"	"	"	I		13.4	13.45	10	850 000	1 071 800						.00036		"
16	"	"	"	"	"	"	"	"	I		13.4	"	10	576 000	996 000	1 009 900	476 360	533 540	39 800	(46 000)† 41 600	.00036		"
17	"	"	84.8	$4-\frac{3}{8}$ $2-\frac{1}{4}$	.66	"	"	"	6	$\frac{3}{4}$	12.37	14.6	12	732 500	951 500						.00037	.0070	"
18	"	"	"	"	"	"	"	"	"	"	"	"	12	630 500	909 000	930 250	405 000	525 250	42 400	(47 000)† 42 400	.00041	.0060	"
19	"	"	"	"	"	"	"	"	"	"	"	"	14	630 500	888 500						.00038	.0100	"
20	"	"	"	"	"	"	"	"	"	"	"	"	14	679 000	827 500	858 000	405 000	453 000	36 650	(40 500)† 36 650	.00033	.0064	4400

Columns and cylinders were 62 to 66 days old when tested. Concrete mixture 1 : 1 : 2. * Probable actual area. † Probable actual load.

PROFESSOR TALBOT'S TESTS ON STRUCTURAL STEEL COLUMNS REINFORCED BY HOOPED CONCRETE.

Outside diameter	16 in.
Core diameter	14 in.
Actual area of concrete within core	143 sq. in.
Vertical reinforcement	none
Spiral reinforcement:	
Diameter and pitch	not mentioned
Percentage of full core area	1 per cent.
Cast iron—Gray type structural steel column:	
Area	13 sq. in.
Percentage of actual concrete area	9.1 per cent.
Length of column	10 ft.
Ultimate load, about	856,000 lb.
Strength of hooped concrete 1 : 2 : 4, probably	3,000 lb. per sq. in.
Load considered carried by concrete	429,000 lb.
Load considered carried by structural steel (427,000 lb.)	32,800 lb. per sq. in.

about 18 in. square, consisting of $1\frac{5}{8}$-in. dressed lumber nailed together with cleats, was carefully leveled; on these wooden plates circles were drawn for the cast-iron columns and the outside wooden forms. The latter consisted of two half cylinders made up of $\frac{7}{8} \times 2\frac{3}{4}$-in. dressed boards nailed to wooden rings about two feet apart. The rings were held together with $\frac{3}{8}$-in. bolts.

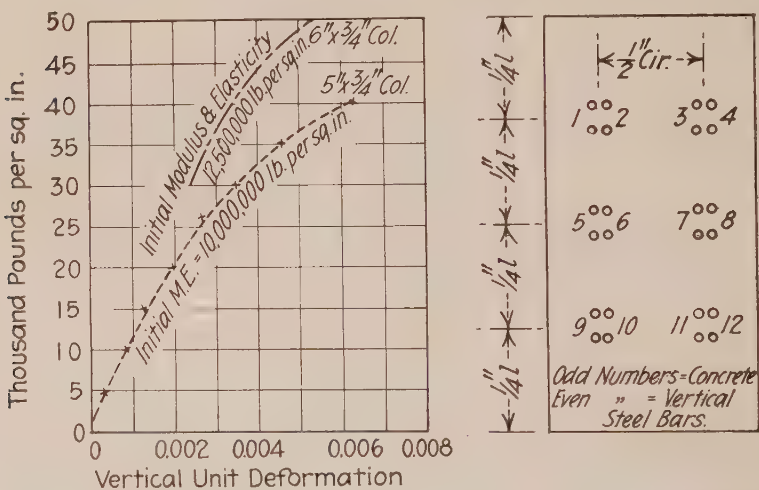

FIG. 6.—TEST OF CAST-IRON COLUMN AND DEVELOPED SURFACE OF CONCRETE COLUMN SHOWING LOCATION OF GAGE POINTS.

Column 5 in. dia., 24 in. high.

Area of cast-iron by planimeter 9.10 sq. in.

Ultimate strength per sq. in. $= \dfrac{662,000}{9.1} = 72,800$ lb.

Upper curve for cast-iron column 6 in. dia., 24 in. high.

Area by planimeter 11.1 sq. in.

Ultimate strength = 89,500 lb.

The cast-iron columns were first placed on the wooden plates and held in position with three nails, then the reinforcing was slipped over the columns and the forms were placed around them and held in exact position at the bottom with four wooden blocks.

The forms were then plumbed and held in vertical position and the cast-iron columns were centered on top by means of 1 x 2-in. cleats nailed to the forms. The reinforcement was in general from $\frac{1}{4}$ in. to $\frac{3}{8}$ in. shorter than the columns. The cast-iron columns were considerably displaced at the lower end; as much as $1\frac{1}{2}$ in. in columns 5, 6, and 13. The concrete was mixed by hand in a mortar box about 4 ft. wide, 6 ft. long and 8 in. high. Each batch consisted of 3 bags of cement, 3 cu. ft. of torpedo sand and 6 cu. ft. of gravel. The sand was first placed in the mixing box and leveled off, then the cement was evenly spread over the sand and the whole turned over with shovels twice by two experienced laborers and leveled off again in the box. To this was added the gravel to a uniform level and the whole again turned over twice and leveled off again. Then about 13 gal. (6 small pails) of water was added by pouring it slowly over the whole surface and the concrete mixed twice, then filled into pails and slowly poured into the columns. The consistency of the concrete would probably be called mushy.

The concrete inside the forms was tamped with a 1 x 2 in. pole. The top of the columns were finished the next day with sand and cement 1 : 1, by means of a piece of plate glass using the top of the cast-iron column as a guide; a perfect end, however, was not obtained. The forms were generally removed the next day and were used from four to five times. The columns were made between Sept. 26, and Oct 2, during which time the temperature varied from 39° to 76° F., with an average of 60°, and were sprinkled three times a day for two weeks. During October the temperature averaged from 50° to 60° F. On Nov. 1, 1916, the columns were loaded on trucks and hauled one block to the railroad crane where they were placed on a car, on the bottom of which 2 x 4-in. strips were laid, supported by blocks about 18 or 20 in. apart to reduce the shocks during transit. The car arrived about one week later at the U. S. Bureau of Standards Pittsburgh Laboratory, and the columns were unloaded and stored on the floor of the laboratory. During the month of November, the temperature to which these columns were exposed was probably 40° to 50°, with the lowest temperature about 10° during transit.

PREPARATION OF COLUMNS FOR TESTING.

It being the intention of making extensometer readings at twelve points of the columns for vertical deformations, it was necessary to cut twelve holes in the columns adjoining vertical rods at diagonally opposite sides at positions shown in Fig. 6; in these holes, two of each were always about 6 in. on centers, $\frac{3}{8}$-in. round iron plugs were fastened by plaster filling, and the outside of the plugs were about even with the inside of the spiral wire. Punch marks were made in these plugs, also in the adjoining $\frac{3}{8}$-in. square bars, from which the extensometer readings were taken by means of a 6-in. Berry gage.

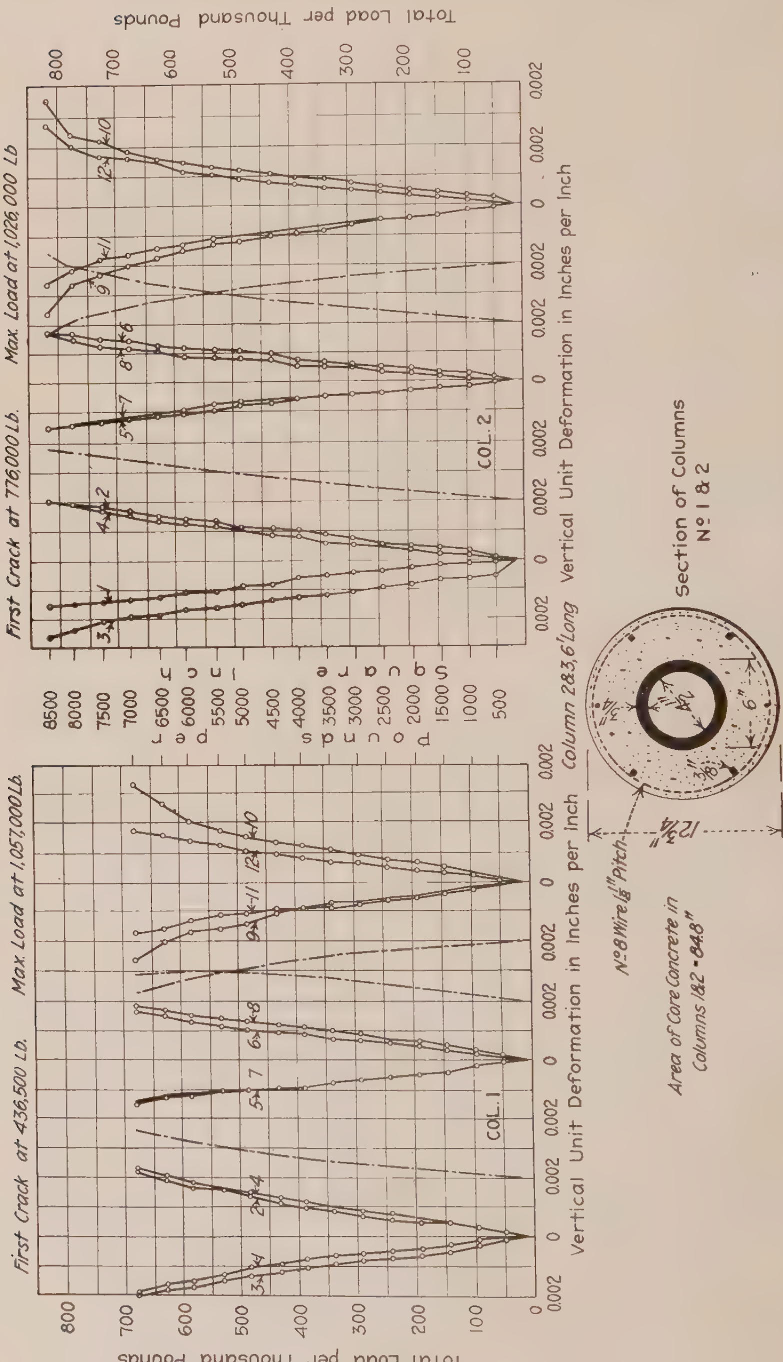

FIG. 7.—STRESS-DEFORMATION CURVES ON COLUMNS 1 AND 2.

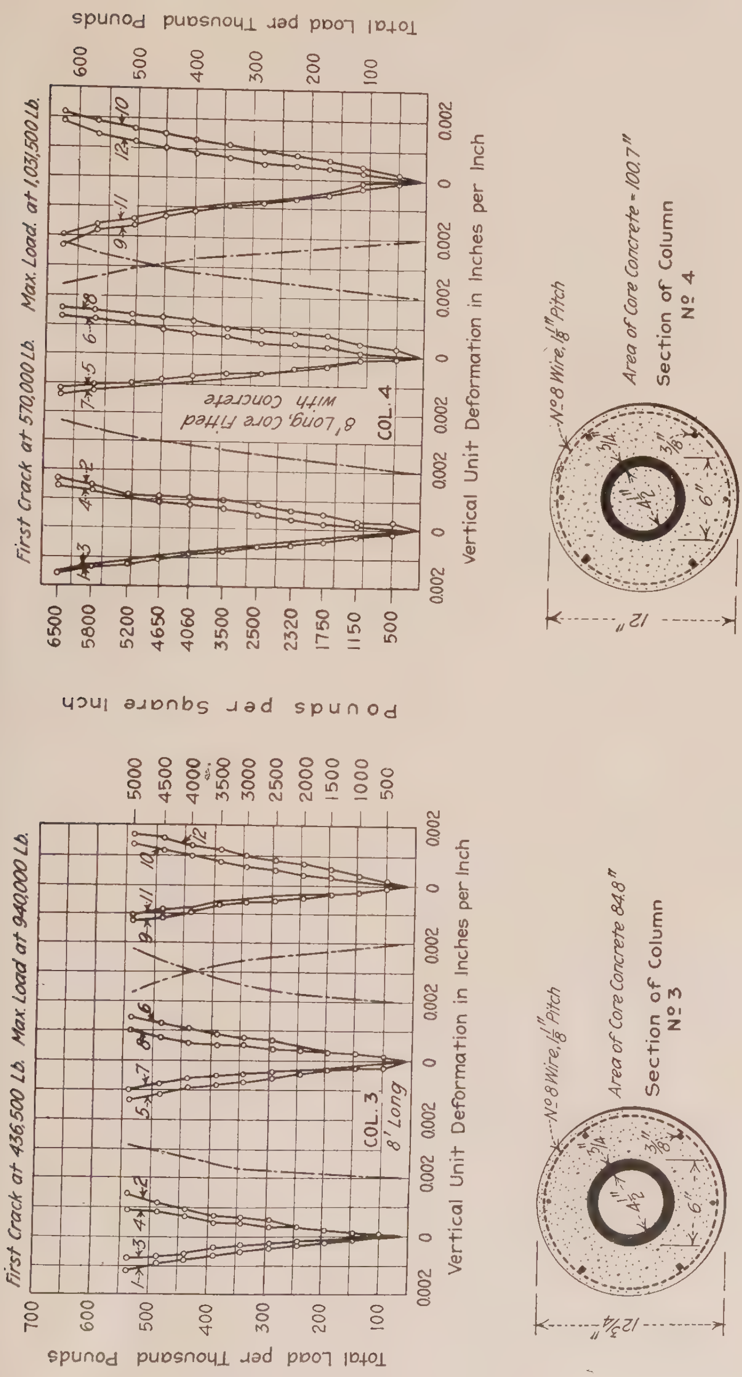

FIG. 8.—STRESS-DEFORMATION CURVES ON COLUMNS 3 AND 4.

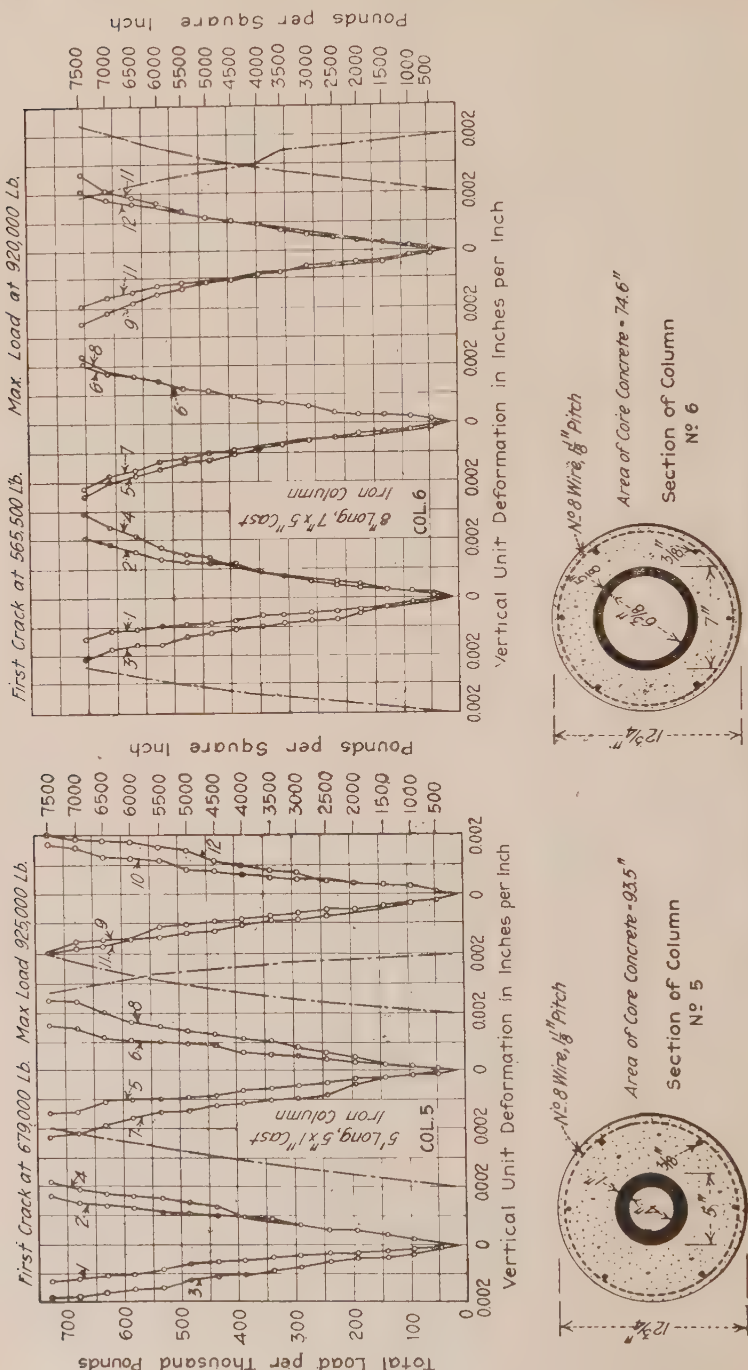

FIG. 9.—STRESS-DEFORMATION CURVES ON COLUMNS 5 AND 6.

TESTING THE COLUMNS.

The columns were tested in the 10,000,000-lb. testing machine of the Bureau of Standards Laboratory at Pittsburgh, Pa. In this testing machine the load is applied by means of a hydraulic piston which is actuated by a hydraulic pump. The load is weighed through a system of levers which are connected with the piston of a smaller hydraulic cylinder which communicates with the larger cylinder. The columns were placed in position for testing after the gage lines had been prepared; a layer of plaster of paris was used on the top and bottom surfaces of the columns. In placing the columns in the machine care was taken to have the upper surfaces of the columns parallel with the upper platen of the testing machine and placed in such a way as to make external excentricity of the load a minimum, then the motor operating the top head was started and the top head was brought down on the soft plaster of paris at the top of the column until the plaster of paris was forced out on all sides.

An initial load of about 10,000 lb. was applied and the initial extensometer reading taken. Besides the extensometer readings at the twelve points 6 in. c. to c., three vertical extensometer readings, called A, B and C, were taken on each column for a length of 60 in. on the 6-ft. columns, 85 in. on the 8-ft. columns, 100 in. on the 10-ft. columns, 125 in. on the 12-ft. columns, and 150 in. on the 14-ft. columns. The apparatus for the latter measurements consisted of two iron rings fastened by screws to the columns in about the correct distance. The upper rings carried three brass tubes about $\frac{3}{4}$-in. in diameter which rested by means of needles on the gages fastened to the lower ring. While the extensometer readings of the 6-in. gage lengths are often erratic due to local conditions of voids, elasticity, eccentricity, etc., the readings at points A, B and C were comparatively very uniform.

The extensometer readings were taken after the pump was stopped and owing to the elasticity of the column, the load slightly dropped off during the five minutes if taken to make the fifteen readings. From the few tests to determine the drop, it is believed that it did not amount to more than 1 per cent at the outside.

The readings were generally continued until the first crack appeared. In order to safeguard the instrument for the A, B and C readings, they were then taken off and also the other readings were discontinued because cracks disturb the plugs and make the readings unreliable.

The columns were 61 to 66 days old when tested. The test cylinders were broken at about the same time and for hand mixing and a rather low temperature showed high and uniform strength.

Fig. 5 shows all the general data of tests; Figs. 7 to 15 show the concrete and cast-iron sections and the vertical unit deformations for the twelve gage points and the A, B and C points of each column. The odd figures represent concrete readings and the even figures steel readings. Opposite concrete readings are always shown to the left of the axis and opposite steel readings to the right. The A, B, C readings are shown by dot-and-dash lines.

Where concrete and steel readings of opposite points vary greatly, we may expect a local eccentricity. Failure occurred though rarely at such points.

HOOPED CONCRETE COLUMNS WITHOUT CAST-IRON REINFORCEMENT.

The first crack occurred at an average load of 465,000 lb. or 4,110 lb. per sq. in., while the concrete cylinders failed at an average of 3,630 lb. per sq. in.

After the first crack the unit deformation of the columns increased faster and the maximum load was evidently reached when the spiral reinforcing was near its ultimate strength. The average maximum load was 540,350 lb. or 4,778 lb. per sq. in., which agrees very well with the formula proposed by the American Concrete Institute, according to which 1 per cent of vertical reinforcement contributes 400 lb. per sq. in., and 1 per cent of spiral reinforcement contributes 1600 lb. per sq. in. In these cases the ultimate strength per sq. in. $=3630+400\times0.58+1600\times0.61=4830$ lb.

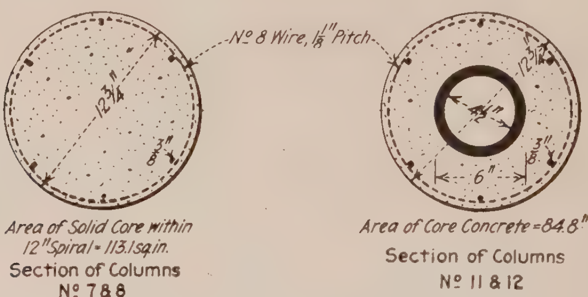

FIG. 10.—CROSS-SECTIONS OF COLUMNS 7, 8, 11 AND 12.

In order to find out how much the cast-iron carried in the other columns, the assumption was made that 4,778 lb. was the strength per square inch of the concrete within the core, and that the remaining load was supported by the cast-iron columns.

COLUMNS WITH CAST-IRON REINFORCEMENTS.

The ten columns 8 ft. to 12 ft. long showed very uniform results. Only in column 3 the first crack appeared at the low load of 436,000 lb., or about one-half the maximum load; in all other cases the first crack was noticed at the loads of 600,000 to 800,000 lb., or at $\frac{6}{10}$ to $\frac{8}{10}$ the maximum load. The unit deformation at this time varied from 0.00125 to 0.00175, practically the same at which the plain concrete cylinders failed.

At a load of about 800,000 lb. considerable spalling of the outer shell occurred. Sometimes the spalling started at the top or bottom, and then the spalling might start in the center and vice versa, clearly indicating that the

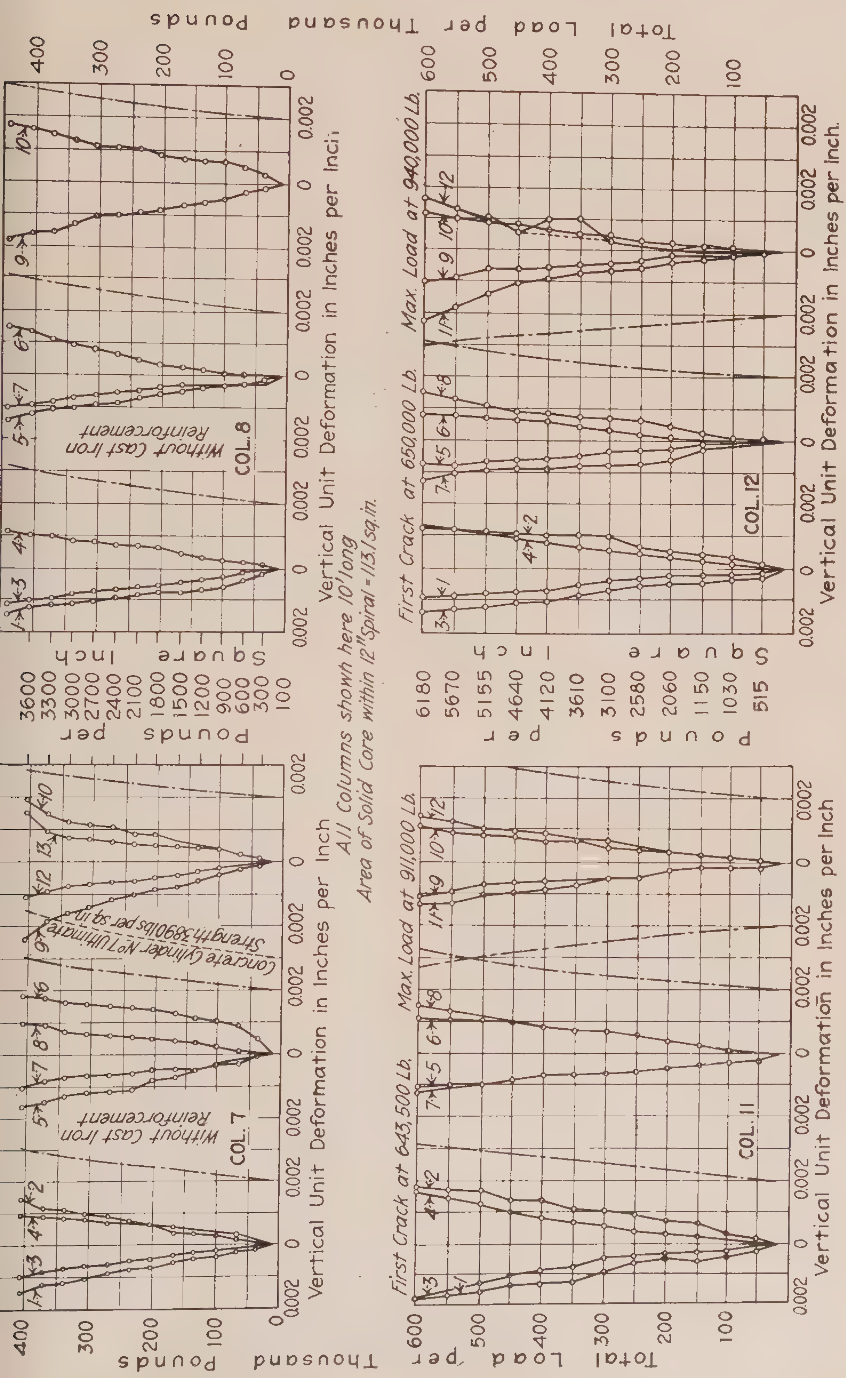

FIG. 11.—STRESS-DEFORMATION CURVES ON COLUMNS 7, 8, 11 AND 12.

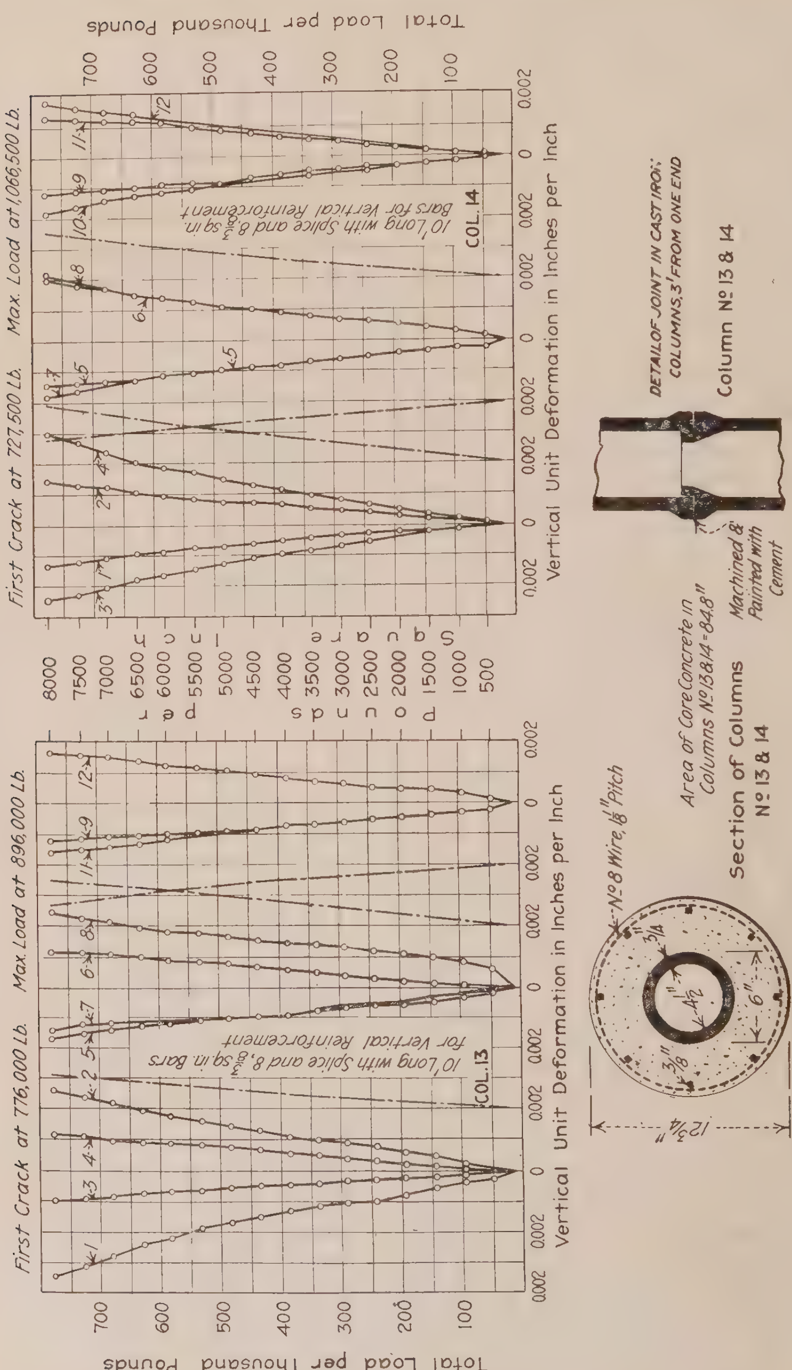

FIG. 12.—STRESS-DEFORMATION CURVES ON COLUMNS 13 AND 14.

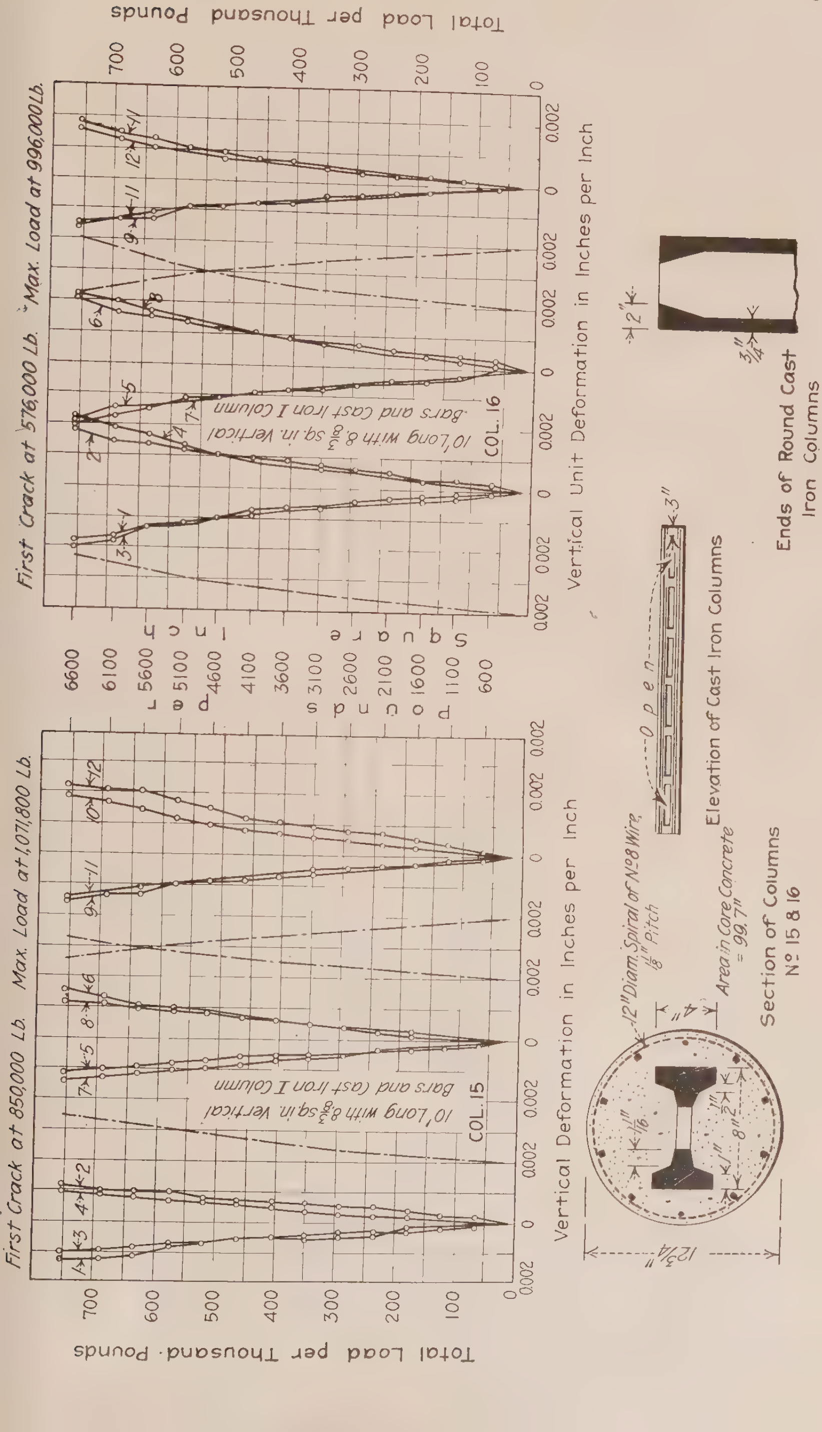

FIG. 13.—STRESS-DEFORMATION CURVES ON COLUMNS 15 AND 16.

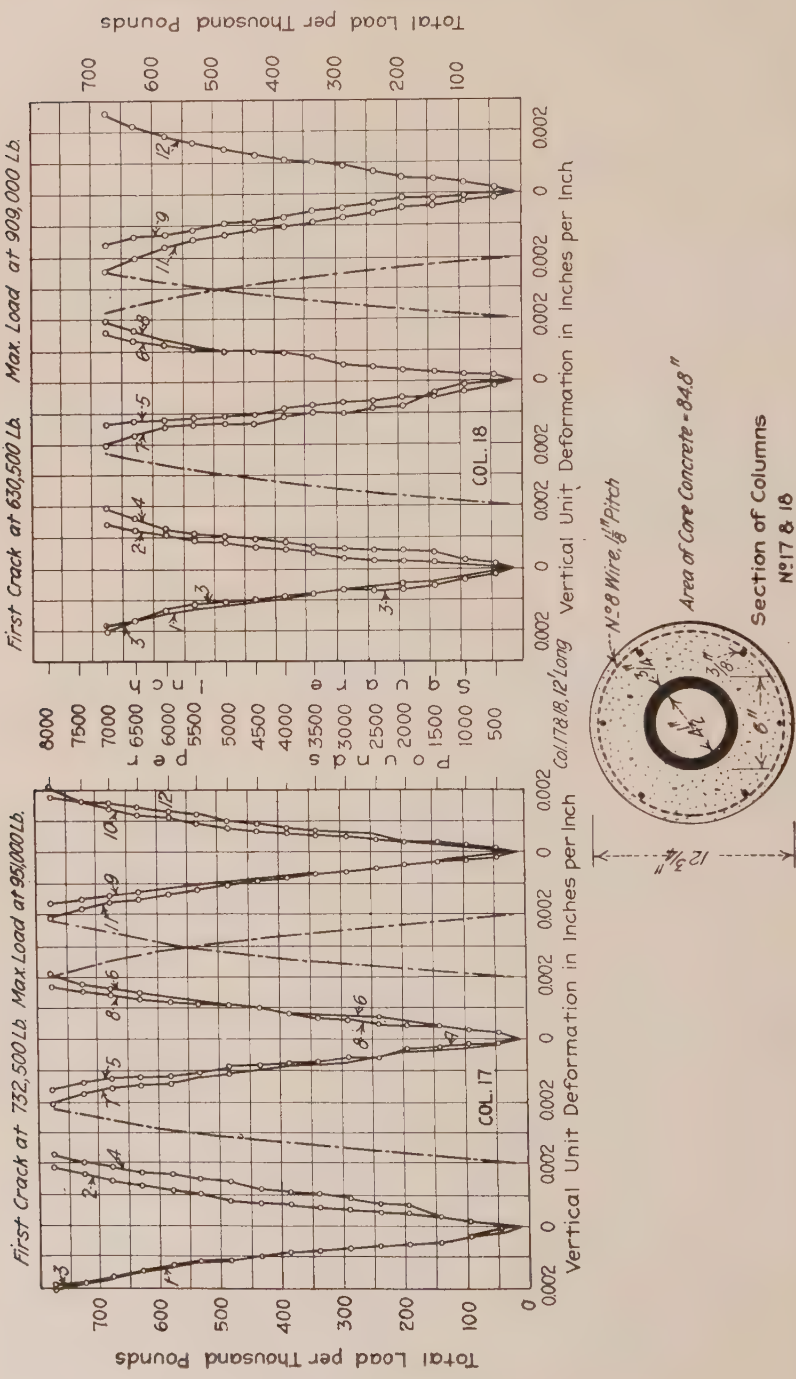

FIG. 14.—STRESS-DEFORMATION CURVES ON COLUMNS 17 AND 18.

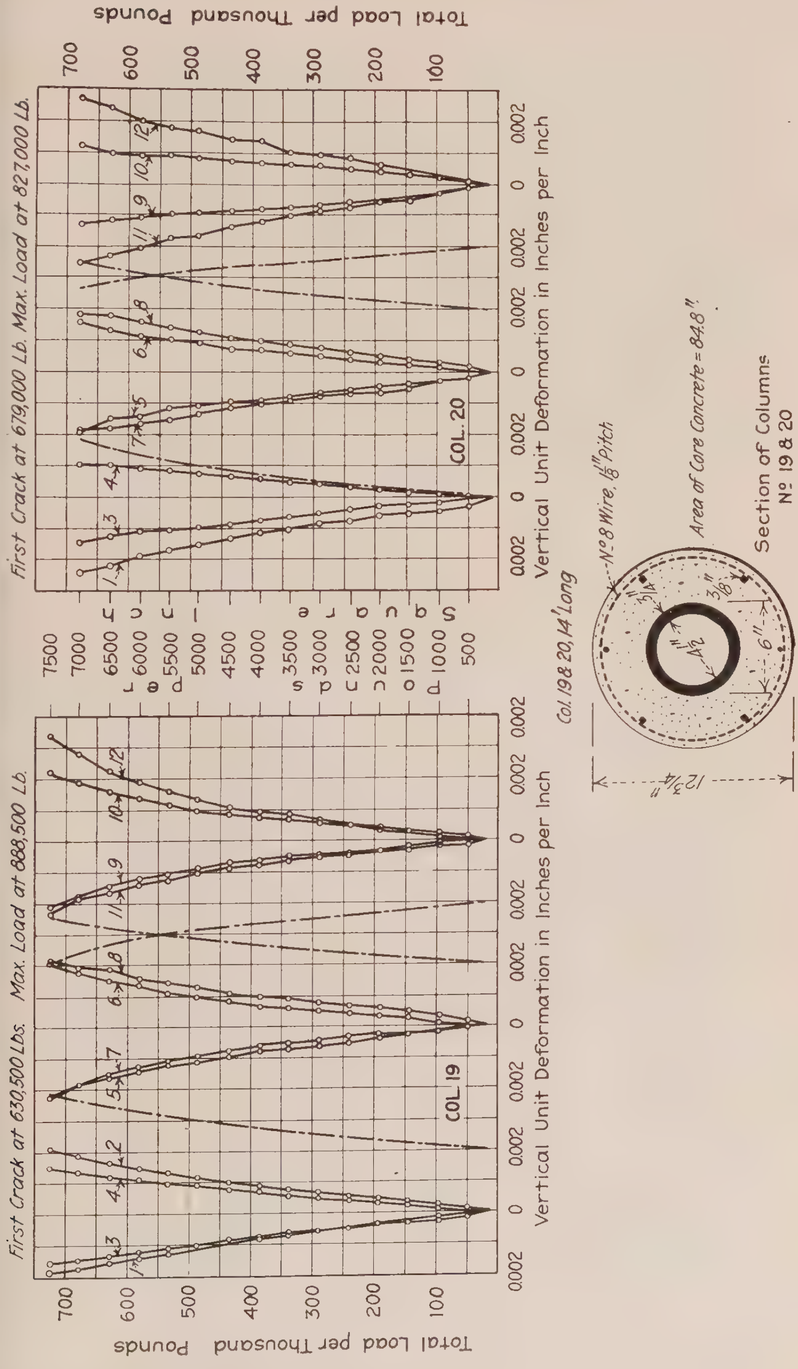

FIG. 15.—STRESS-DEFORMATION CURVES ON COLUMNS 19 AND 20.

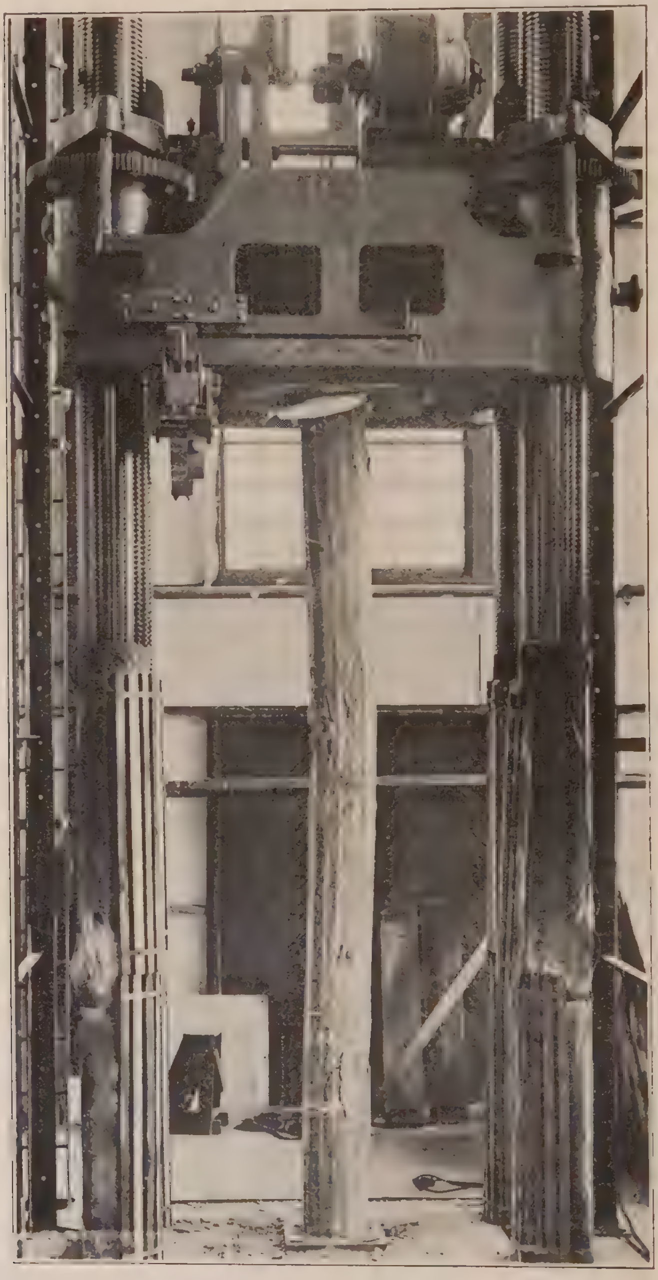

FIG. 16.—CAST-IRON AND CONCRETE COLUMN IN TESTING MACHINE.

cast-iron column helped to carry the stresses over the weak spots in the concrete.

Near the ultimate load, sometimes one or two spirals burst, but still the cast-iron columns helped to distribute the load, and, the spirals often burst in quite distant places.

When the pump was working after the maximum load was reached, the concrete where the spiral burst was destroyed and after a few minutes fell out and exposed the cast-iron. Even then the cast-iron probably carried 80 per cent of the maximum load.

In two cases where the pump was kept running perhaps 10 min. after the maximum load was reached and the cast-iron was exposed for about 6 in., circular cracks appeared around the columns about 12 in. on centers, which showed the tendency of the cast-iron to transmit the load by bond again into the undamaged part of the concrete, whereby the bond between the cast-iron and the concrete was overcome.

In column 4 the core of the cast-iron column was filled with concrete and showed a gain proportionate to the extra concrete.

Cast-iron columns 13 and 14 were spliced 3 ft. from the end as shown in Fig. 12. The average load carried by these two columns was 981,000 lb., while columns 11 and 12 without splices averaged only 925,000 lb. Columns 13 and 14 had 0.6 sq. in. of extra vertical steel which would account for not more than 24,000 lb. of additional strength, so that it may be safely stated that splices do not diminish the strength of such columns.

Columns 15 and 16 had I-sections as reinforcement as shown in Fig. 13. The object was to find out whether a smaller radius of gyration in one direction can be counteracted by the stiffness of the concrete. These columns carried practically the same unit stresses as columns 11 and 12 with round cast-iron columns.

The use of such sections may be of advantage in cases where continuous girders are attached to the columns, giving the girder bars an opportunity to pass by, or in high buildings they may be used for outside columns where great stiffness is required in one direction.

From the fact that the maximum loads of all columns were attained when the spiral reinforcement was near the breaking point, we may infer that higher ultimate values of the strength of these columns would have been obtained if a larger percentage of spiral reinforcement had been used.

To sum up, we have in hooped concrete columns reinforced with cast-iron, a new type of compression member which can sustain stresses up to 17,000 lb. per sq. in., and hence allows smaller sizes than columns built of structural steel, at a very great saving in cost.

DISCUSSION.

Mr. Thompson.

Mr. S. E. Thompson.—I have long considered cast-iron as theoretically very fine reinforcement because of its low modulus of elasticity and consequently the greater strength that could be obtained in combination with concrete. The great difficulty however, and it seems to me the insurmountable difficulty in its adoption, is its unreliability. I would like to ask if these tests today have not shown that point? The columns broke from flaws in the cast-iron, and this would prohibit its general use as structural material.

Mr. Mensch.

Mr. L. J. Mensch.—The columns attained the ultimate load long before the cast-iron broke, and all the breakages are due to running the machine about 10 minutes after the maximum load was obtained. The cast-iron was of extremely poor grade; it was made of 50 per cent of scrap and 50 per cent of cast-iron and contained dirt and blowholes in it, and the eccentricity in the columns was as much as 1 in.

Mr. Conzelman.

Mr. John E. Conzelman.—In such columns as these it has always been a problem in my mind how to get load to the core. In a building with such columns how could you get the load from the floor to the column? Would you have cast-iron brackets or how would you make the connection?

Mr. Wight.

Mr. Frank C. Wight.—In connection with Mr. Conzelman's question I would like to call attention to a well-known concrete failure about three years ago which throws some light on the problem. There was an old building which adjoined a reinforced-concrete building about to be built. The design called for a reinforced-concrete column at the connection between old and new work. When they began to tear down the building, they found a cast-iron column there and, without authority, the building superintendent proceeded to put a 3-in. coating of concrete around this cast-iron column and to frame the connecting reinforced-concrete floor-beam into this annular ring with no reinforcing tie. The rather deplorable consequence was that when he pulled his forms the beam fell and the whole building collapsed. So far as anybody could find out, the whole bearing was this 3-in. annular ring of concrete around the cast-iron column.

Mr. Mensch.

Mr. L. J. Mensch.—I remodeled, about 15 years ago, some posts with cast-iron columns and did the same thing Mr. Wight mentioned, and I never had any trouble. It is not good practice, however, to try to put a very great load on the shell around a cast-iron column. In the columns under test it is not proposed to make the ring less than 5 in. thick.

EXTENSOMETER MEASUREMENTS IN A REINFORCED-CONCRETE BUILDING OVER A PERIOD OF ONE YEAR.

By Arthur R. Lord.*

From the time of the writer's first extensometer test of a reinforced-concrete building in 1910 this method of studying the structural action of floors and columns under load has found ever-increasing application, being especially important in connection with the flat-slab type of concrete construction in which any satisfactory mathematical treatment is still lacking. A great many tests of this character have been made to date, although a considerable part of the data remains unpublished. In all of the tests previously made and reported, so far as I am informed, the loading has been progressively increased until a load considerably in excess of the design load was reached, and this load held in place approximately 24 hours and then released. None of the tests, I believe, have involved extensometer measurements of deformations in the steel and concrete under a heavy test load held in place through a long period of time.

The ordinary 24-hour extensometer test has afforded us some exceeding valuable information, both on the general problem of design and on the handling of details. Such tests, however, do not inform us as to the effect of temperature changes on the structural action of buildings nor as to the effect of long-continued or repeated loading. In 1912 the writer in connection with the test of the Franks Building in Chicago (the regular 24-hour extensometer test of this building was made under direction of Prof. W. K. Hatt of Purdue University) attempted by extensometer measurements to observe the effect of temperature changes in the test floor. The building, however, was in daily use and the rapidly shifting load conditions very thoroughly obscured the desired indications so that no conclusions seemed warranted and the readings were discontinued.

An excellent opportunity to get at one of these problems was presented by the test of the Schwinn Building, described in this paper. This building was designed by Lieberman & Klein, Consulting Engineers, shortly before the present Chicago Flat-Slab Ruling went into effect. At that time each designer was required to satisfy the building department as to the sufficiency of his design either by previous tests of similar constructions or by submitting the proposed building to an extensometer test prior to its occupation. The writer was employed by the city as testing engineer on this occasion, and it was at his suggestion, with the hearty consent of the designing engineers, that the test load of twice the designed dead- and live-load was required to remain in place one full year. The ordinary extensometer test was first made in the usual manner as has been described before this Institute many times, readings

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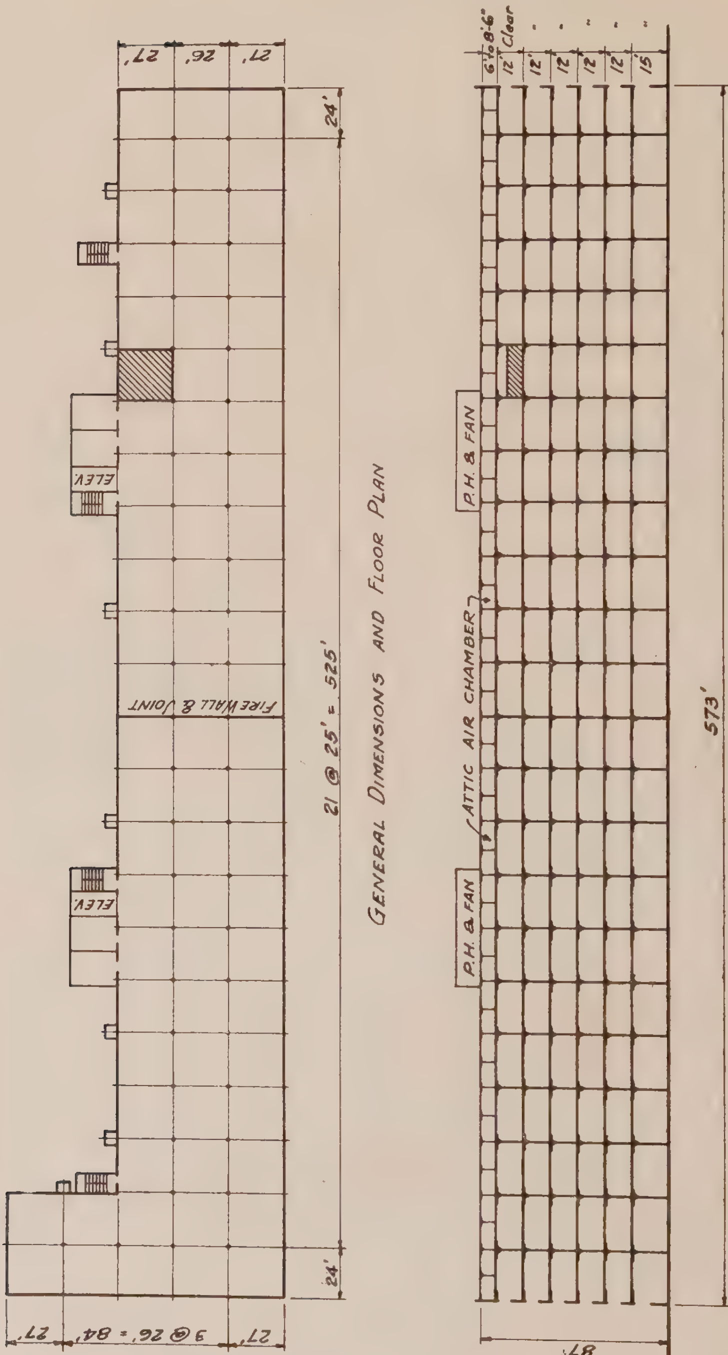

LONGITUDINAL SECTION

FIG. 1.

of deformation being taken under various degrees of loading and a final reading being taken when the full test load was in place 24 hours. From that point 44 gage lines of representative character and showing the greatest stresses of their kind and location were selected and these lines were measured during the further period of slightly over one year. The results are reported herewith:

THE BUILDING TESTED.

The Schwinn Building, occupied by the Excelsior Motor Manufacturing and Supply Co., is located at Lawndale Avenue and Cortland Street, Chicago, Ill. As indicated in the sketch, Fig. 1, the building is six stories and attic in height, approximately 673 ft. long by from 80 to 138 ft. wide. The building is divided at the center by a fire wall and an expansion and contraction joint was made at this point, the slab being designed to slide on the same beam that supports the wall. The panel tested was selected by Mr. Jensen of the Chicago Building Department. In view of the size of the panel (approximately 25 x 26 ft.) and the amount of test load involved only one panel was loaded. The test panel is hatched in Fig. 1. It was located on the sixth floor where the columns were the smallest and the effective span the greatest. There were two flat slabs above the slab tested and four below. The elevators, toilets and wash-rooms for the building were located in service wings and one of these projected from the panel immediately adjoining the test panel. The attic was used as an air storage and circulation chamber and was connected with the various stories by means of concrete and metal ducts. The concrete ducts were built on the outside of certain wall columns and the metal ducts carried through holes in the floor slab adjoining each wall column. One concrete duct adjoined the test panel and openings for two metal ducts were present at each of the wall columns of the test panel.

DETAILS OF TEST PANEL.

The details of the slab reinforcement are given in Fig. 2. One-third of the slab rods in each direct band were carried through at the column and lapped, without being bent up to take tension. All the diagonal bars were bent up. The slab was designed to be $9\frac{1}{2}$ in. thick, but actually measured a uniform 10 in. in thickness practically throughout the area of the building. The slab steel at the column was not raised to take advantage of this increased thickness, however. The interior drops were 9 ft. square at their lower surface and 6 in. thick. The edges were beveled to a 10 ft. 6 in. square at the under surfaces of the slab. The wall drops were of corresponding size but only 4 in. thick. The exterior walls were of concrete standing 3 ft. above the floor level, and 9 in. thick. The wall columns were 6 ft. 6 in. wide by 1 ft. 6 in. thick with a pilaster 2 ft. 6 in. wide by 1 ft. thick at the center. They were reinforced with sixteen $\frac{3}{4}$-in. round rods 14 ft. 6 in. long and tied at intervals of nine inches with $\frac{1}{4}$-in. round wire.

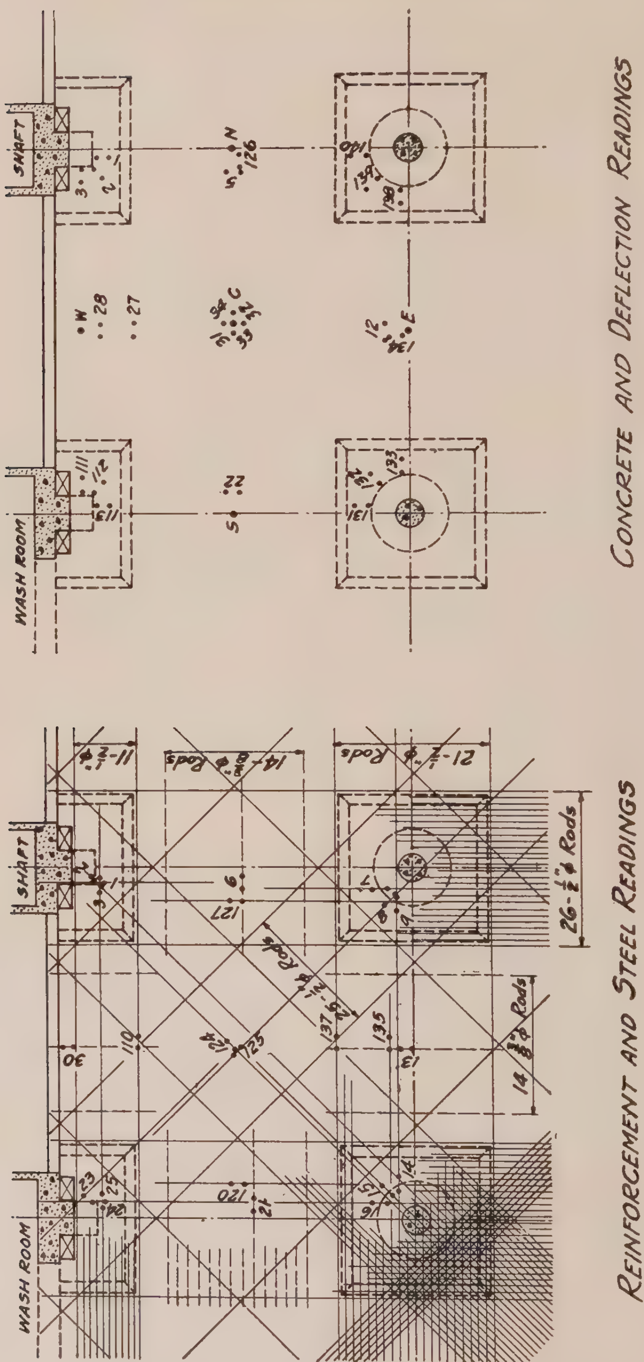
WASH ROOM
SHAFT
CONCRETE AND DEFLECTION READINGS
WASH ROOM
SHAFT
REINFORCEMENT AND STEEL READINGS

FIG. 2.

The interior columns were built as follows:

C-18, 5th story.
28 in. dia., 24 in. spiral $\frac{1}{2}$ in. round at $2\frac{1}{4}$ in., eleven 1-in. sq. rods.
C-18, 6th story.
26 in. dia., 22 in. spiral $\frac{3}{8}$ in. round at 2 in., eight $\frac{7}{8}$-in. sq. rods.
C-19, 5th story.
26 in. dia., 22 in. spiral $\frac{3}{8}$ in round at 2 in., eight $\frac{7}{8}$-in. sq. rods.
C-19, 6th story.
26 in. dia., 22 in. spiral $\frac{3}{8}$ in. round at 4 in., eight $\frac{3}{4}$-in. sq. rods.

The concrete in the test panel, columns and wall was of 1 : 2 : 4 mixture. The columns were poured on June 12, 1914, and the test panel on the following day. The wall was poured to the sill level about 3 hours after the slab was placed and the wall columns were poured to the same height at the same time. The cement floor finish (from 1 to 2 in. thick) was laid on the following day, the total thickness of slab and finish being 10 in. The top steel was placed after the rough floor slab concrete had stiffened considerably and was from $1\frac{1}{4}$ to $1\frac{3}{4}$ in. below the top of the slab. The structure was "fire-proofed" with not less than 1 in. of concrete over and under all slab steel.

AUXILIARY TEST SPECIMENS.

Concrete specimens in the form of 6-in. cubes, 8 x 8 x 16-in. prisms and 6 x 8 x 54-in. control beams were taken at random from the concrete actually placed in the test panel. Cubes and prisms were shipped to the University of Illinois for testing, one-fourth at the time of the 24-hour test, and one-fourth at 4 months, 8 months and one year thereafter. The control beams were tested at the building at the time of the 24-hour test. All specimens were stored on the floor up to a few days before the test began.

TABLE 1.—TESTS OF CONCRETE SPECIMENS.

Age.	Compressive Strength.		Modulus of Elasticity, Prisms, lb. per sq. in.
	Cubes, lb. per sq. in.	Prisms, lb. per sq. in.	
67 days	2420	2150	3,700,000
About 6 months	3180	2570	3,500,000
About 10 months	3080	2733	4,350,000
About 14 months	2100*	2500	3,400,000

* One specimen chipped at edge.

Control beams loaded at the third points on a 4-ft. span gave a modulus of rupture for two specimens of 181 and 190 lb. per sq. in.

TEST LOADING.

The first load was placed on the test floor on Aug. 14, 1914, amounting to an average depth of 22 in. of bank sand (about 160 lb. per sq. ft.). On Aug. 15th, the load was increased to an average depth of 38 in., very well

compacted. The car weights gave the total load as 192,000 lb. (about 300 lb. per sq. ft.). On Aug. 17th the load was finally increased to an average depth of about 57 in., the total car weight being 289,600 lb. (about 460 lb. per sq. ft.). Allowing for a small loss in handling the sand from the cars to the test floors, the test load may be considered to have been 450 lb. per sq. ft. and this load remained in place until Sept. 2, 1915, a period of somewhat over one year. The load was necessarily reduced at various points where shelters were built to enable observers to work but the sand was piled higher and thoroughly compacted at these points and the resulting load condition is believed to have been not far from uniform. This floor was designed for a uniformly distributed live load of 150 lb. per sq. ft.

TEST READINGS.

To and including the completion of the 24-hour test (full test load in place for 24 hours) 90 gage lines on the steel and concrete were read continuously with an University of Illinois type of Berry extensometer. Temperature variations in the instrument were corrected by reading on an invar steel standard bar. For the continuation test of one year only 44 gage lines were read, the lines showing the highest stresses at the 24-hour period being selected for this purpose, including, of course, representative gage lines of both steel and concrete in all parts of the floor. The gage lines are located in Fig. 2. Deflections readings (location also shown in Fig. 2) were established at the panel center and near the center of each edge. The readings reduced to unit deformations are plotted in Figs. 3 to 10.

DEFLECTIONS READINGS.

The deflection at the panel center is plotted in Fig. 3. The matter of devising an apparatus for measuring deflections that should be proof against damage from men working in the building was not satisfactorily solved until sometime after the test began. There are gaps in all the deflection readings for the edges of the panel and it was only by virtue of taking two independent series of observations by different methods that a gap in the panel center deflection record was avoided. Both series were broken but not in any one interval.

The test required that several panels on both the sixth and fifth floors should not be occupied for manufacturing and storage purposes, and while no friction developed to the writer's knowledge several events transpired to interrupt the smooth working of the program planned. The extensometer measurements were not affected by such interruptions but merely the regularity in taking the readings. No part of the sixth floor for several panels around the test load was used during the year (except for light trucking) and it is believed that the effect of loading adjacent panels is not involved in this test.

A study of Fig. 3, comparing the temperature and deflection curves for the year, would indicate (1) a general tendency for the deflection to increase

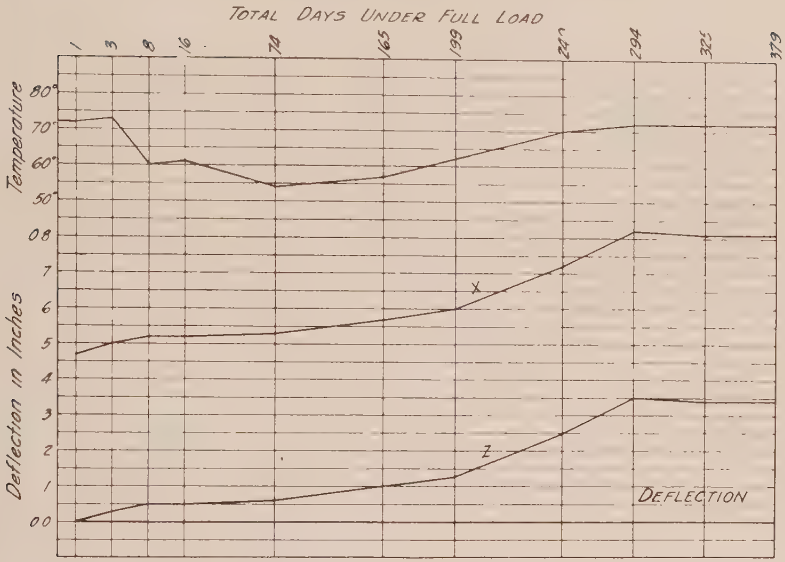

FIG. 3.—TEMPERATURE AND INFLECTION CURVES.

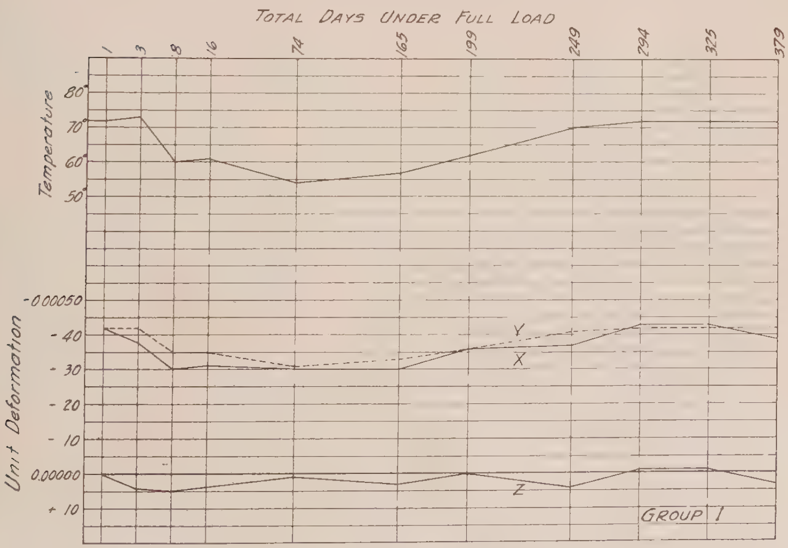

FIG. 4.—TENSILE DEFORMATION IN STEEL AT WALL COLUMN HEAD.

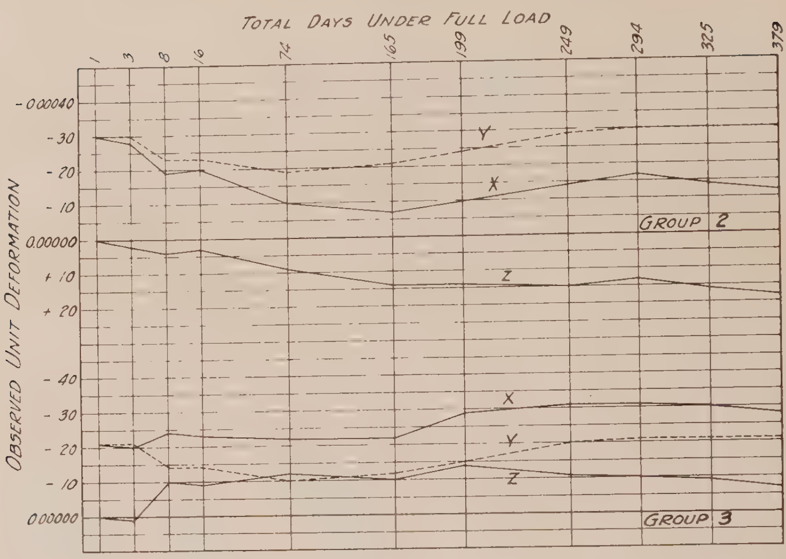

FIG. 5.

GROUP 2—TENSILE DEFORMATION IN STEEL AT INTERIOR COLUMN HEAD.
GROUP 3—TENSILE DEFORMATION IN TOP RODS.

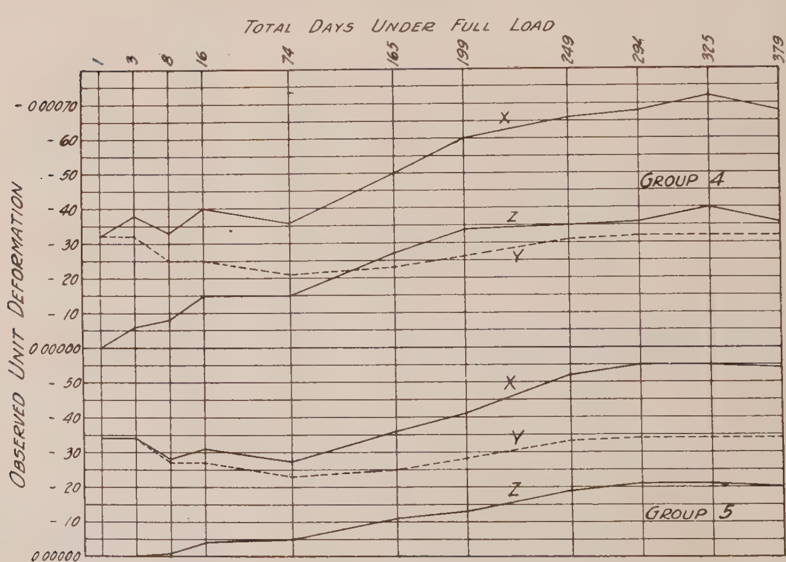

FIG. 6.

GROUP 4—TENSILE DEFORMATION IN STEEL AT CENTER OF DIAGONAL BANDS.
GROUP 5—TENSILE DEFORMATION IN STEEL AT CENTER OF DIRECT BANDS.

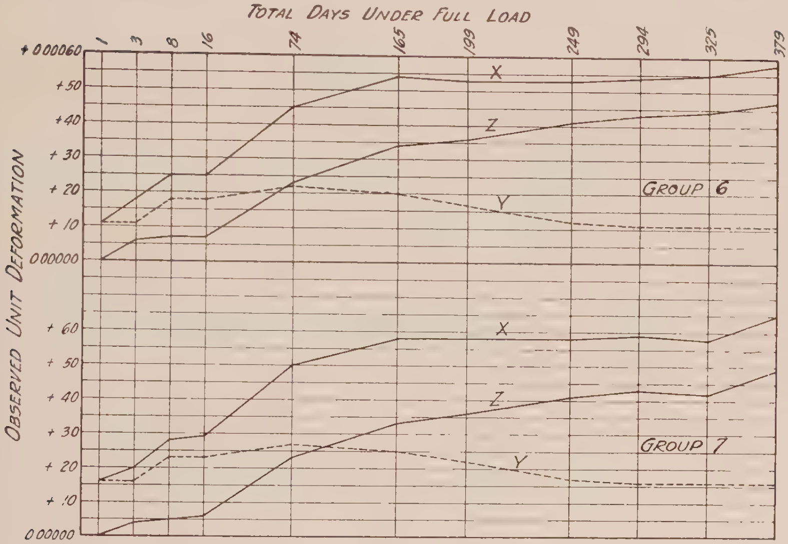

FIG. 7.

GROUP 6—COMPRESSIVE DEFORMATION IN CONCRETE ADJOINING WALL COLUMN CAPITAL. GROUP 7—COMPRESSIVE DEFORMATION IN CONCRETE ADJOINING INTERIOR COLUMN CAPITAL.

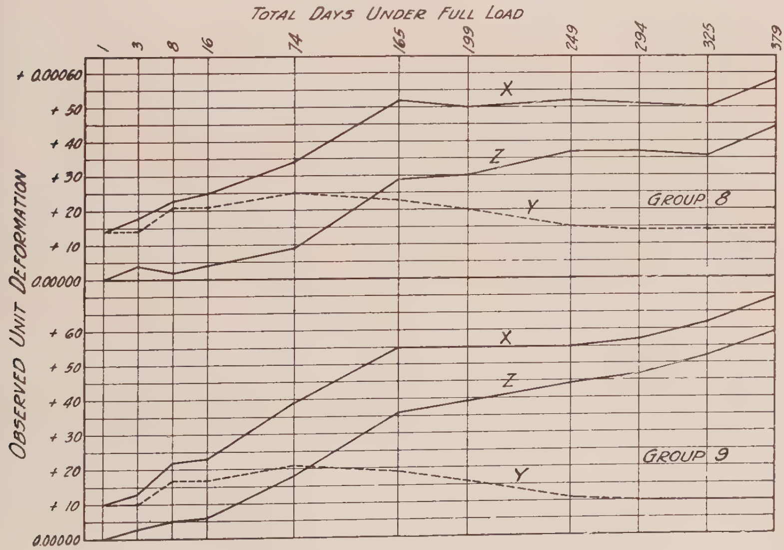

FIG. 8.

GROUP 8—COMPRESSIVE DEFORMATION IN CONCRETE AT DIAGONAL CENTER OF PANEL. GROUP 9—COMPRESSIVE DEFORMATION IN CONCRETE AT CENTER OF DIRECT BANDS.

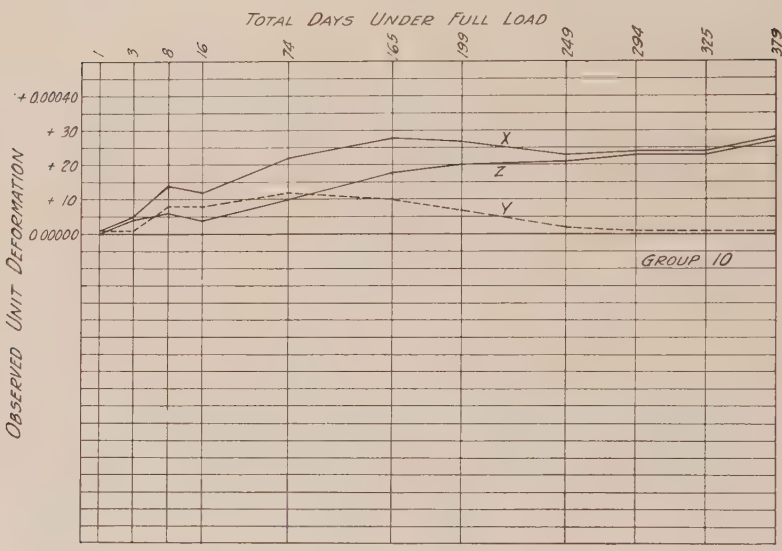

FIG. 9.—COMPRESSIVE DEFORMATION IN CONCRETE ACROSS DIRECT BANDS AT CENTER.

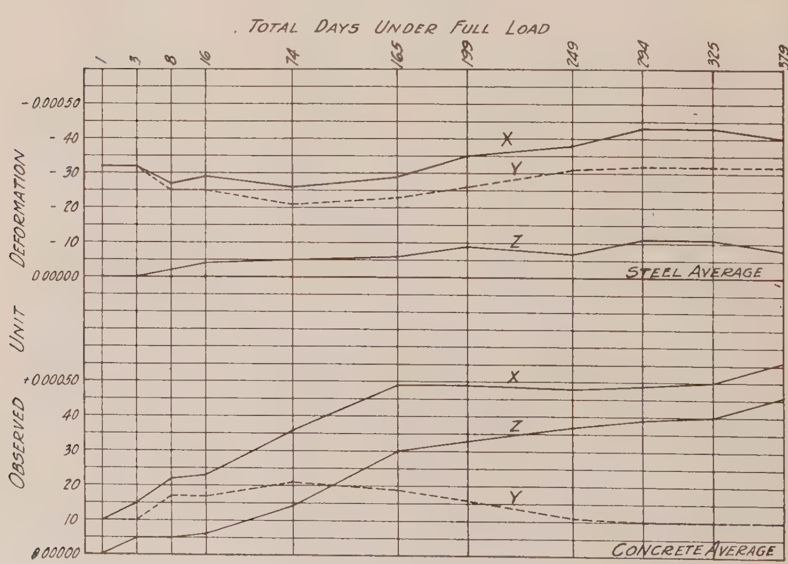

FIG. 10.—AVERAGE DEFORMATIONS IN STEEL AND CONCRETE.

at all times during the year and especially during the earlier stages of adjustment, due, I presume, to the plastic yielding of the concrete and (2) a general tendency for the deflection to decrease as the temperature falls and to increase as the temperature rises. In the first 74 days when the temperature was falling the natural increase in deflection with time was very largely prevented by the tension brought upon the floor due to the falling temperature. From 74 days to 294 days the rising temperature and standing load were *both* acting to increase deflections. In the final three months of practically constant temperature the deflection remained nearly constant.

GROUPING OF TEST OBSERVATIONS.

Figs. 4 to 10 show the average observed unit deformations in the steel and concrete arranged in groups of similar observations as follows:—

Group 1—Tension in steel at wall column head.
" 2—Tension in steel at interior column head.
" 3—Tension in top rods.
" 4—Tension in steel at center of diagonal bands.
" 5—Tension in steel at center of direct bands.
" 6—Compression in concrete adjoining wall column capital.
" 7—Compression in concrete adjoining interior column capital.
" 8—Compression in concrete at diagonal center of panel.
" 9—Compression in concrete at center of direct bands.
" 10—Compression in concrete across direct bands at center.

The groups are composed of the following gage lines (see Fig. 2 for location):—

Group 1—Nos. 1, 2, 3, 23, 24, 25.
" 2— " 7, 8, 9, 14, 15, 16.
" 3— " 6, 13, 21, 30.
" 4— " 124, 125.
" 5— " 110, 120, 127, 135, 137.
" 6— " 101, 102, 103, 111, 112, 113.
" 7— " 131, 132, 133, 138, 139, 140.
" 8— " 31, 32, 33, 34.
" 9 " 5, 12, 22, 27, 28.
" 10— " 126, 134.

The actual observed unit deformations for the various groups are shown by the "X" curves in Figs. 4 to 10. These are corrected for temperature changes in the extensometer but not for temperature changes in the floor itself. If we consider Fig. 4 for instance, comparing the X curve (actual unit deformations) with the temperature curve we note that the two are generally similar. When the temperature falls the steel stress decreases and when the temperature rises the steel deformation increases. This is contrary to what our reason tells us should be the case, and it becomes necessary to make corrections for the change in temperature of the floor itself.

TEMPERATURE EFFECT ON DEFORMATIONS.

If we conceive of a loaded reinforced-concrete slab resting on a frictionless support and further consider that the temperature falls 10° F. we know that while the stress in the steel remains practically unchanged the measured unit deformation of the steel would decrease approximately 0.00006, the coefficient of expansion for reinforced concrete being approximately 0.000006 per degree F. In other words, while the moment of the external loads and the calculated steel stress remain unchanged the actual unit deformation will be decreased by the contraction due to the change in temperature. If, however, the slab is a part of a reinforced-concrete building the contraction will be resisted by the stiffness of the columns and the actual decrease in unit deformation will be less than 0.00006. The *significant figure* is not the actual change in unit deformation, but rather the difference between the actual change and the amount 0.00006 which would have occurred had the slab been free to contract. This difference measures the amount of the resistance to contraction and gives the actual change in stress. In Figs. 4 to 10 the unit deformation that would have occurred in a body free to contract is plotted in the curves marked "Y," being calculated from the observed unit deformation at the beginning of the test, the change in temperature and the coefficient of expansion of reinforced concrete, 0.000006. These Y curves represent the unit deformations that should have occurred due to the change in temperature alone and neglecting the resistance to free contraction afforded by the columns and also the effect of other tendencies due to long-continued loading that will be discussed later. The *difference* between the X curve and the Y curve is the change in unit deformation that measures a change in actual stress (neglecting changes in the modulus of elasticity). These differences are plotted in the "Z" curves of Figs. 4 to 10 and these Z curves are the final data of this test.

EFFECT OF LONG-CONTINUED LOADING.

Laboratory tests of concrete specimens under long-continued loading have been made since this test of the Schwinn Building, and the results of some of those tests are very valuable in interpreting the data of this test. At the time of making this test the effects of long-continued loading on concrete deformations were not appreciated and tests of auxiliary specimens under long-continued loading were not made. The test itself, however, shows these effects in a very striking manner in Figs. 7 to 10. In Vol. XII of the *Proceedings* of the American Concrete Institute, p. 302, Professors Fuller and More discuss "Time Tests of Concrete," giving data of which I have made use in Fig. 11 to interpret my own test data. Under a constant stress of 1150 lb. per sq. in. these tests of concrete cylinders showed an increase in observed deformation from 21.9 units to 124.5 units in 274 days (Table II, page 303, *ibid.*), while under a constant stress of only 650 lb. per sq. in. these tests showed an increase in observed deformation from 15.0 units to 42.1 units in 52 days (Table III, page 304, *ibid.*). The increases observed in the test of the Schwinn Building were of about the same magnitude. Other tests of less duration have been made which show this same plasticity of concrete.

The direct effect of this phenomena is to render futile any attempt to state stresses in the concrete from the extensometer measurements of the deformations in the concrete in this test. It is possible to indicate the approximate range within which the concrete stresses probably lie. To a much less extent it is undoubtedly true that concrete stresses, as derived from observed deformations in ordinary 24-hour extensometer tests of buildings, are higher than their true values, since in these tests from three to six days are spent in placing the increments of load, and that amount of time is sufficient, as shown by Fuller & More's tests, for a considerable reduction in the modulus of elasticity of concrete in compression. There is a secondary effect which is also important and which was strikingly illustrated in the test

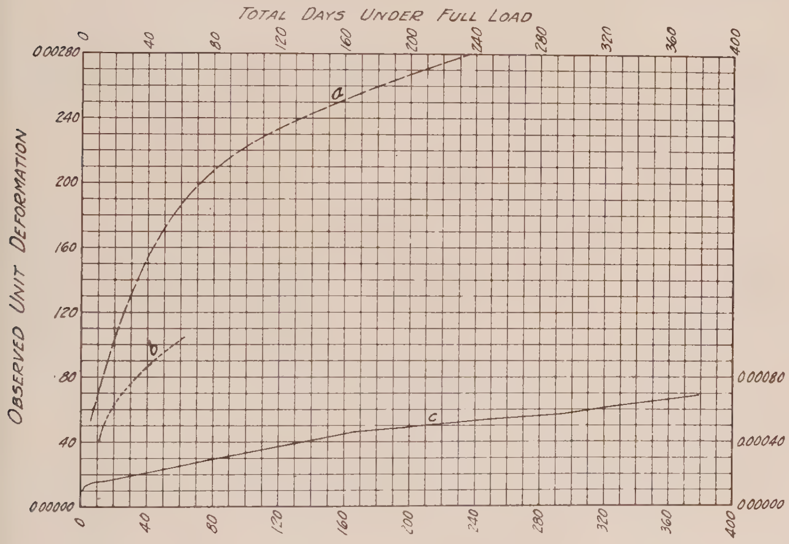

FIG. 11.—RELATIONS OF UNIT DEFORMATIONS TO TIME UNDER LOAD.

of the Schwinn Building. In this test, as the concrete in the lower part of the drop and slab adjoining the columns and in the top of the slab at the center slowly deformed under practically constant stress, an increase in the deflection and stresses at the center necessarily followed. In other words, there was a marked shifting of the moment, increasing at the center as shown by Fig. 6 and decreasing at the support as shown by Groups 1 and 2, Figs. 4 and 5.

SUMMARY OF ACTIONS IN TIME TEST.

To sum up briefly the foregoing, without attempting an exhaustive statement, it seems to me that the principal tendencies to action operating in this test are as follows:—

(1) A stress induced by the resistance of the columns to the movement of the slab under temperature changes. A fall in temperature results in

(*a*) an increase in steel tensions.
(*b*) a decrease in concrete compressions.
(*c*) a decrease in deflections.

(2) An increase in the deformations in the concrete due to the plasticity of concrete under long-continued loading, the increase being more rapid at first and gradually decreasing in rate.

(3) A shifting of the moment due to (2) resulting (for the conditions of this test) in a decrease of the moments at the supports and an increase in the moment at the center.

These three tendencies do not, of course, always act in the same direction or to the same result, and it is interesting to study the effect of the various actions as shown by the plotted test data (Z curves). (2) and (3) apparently dominate the situation. It is also significant that all the actions tend to increase the steel stress at the center under the condition of falling temperatures.

DISCUSSION OF CURVES.

A brief statement of what the Z curves show graphically as to the deportment of the deformation at various points may be useful here.

Group 1 (Fig. 4) and Group 6 (Fig. 7).—Steel and concrete immediately adjacent to wall column capital—The tension in steel shows very little change through the year, the tendency being for the stress to decrease. The compressive *deformation* of the concrete shows a big increase. During the first 16 days the increase was very materially offset by the decrease resulting from the falling temperature.

Group 2 (Fig. 5) and Group 7 (Fig. 7).—Steel and concrete immediately adjacent to interior column capital—The stress in the steel shows a material decrease during the first 5 months of the test, with very little change in the last 7 months. The compressive deformation behaved much as Group 6 above.

Group 3 (Fig. 5) and Group 10 (Fig. 9).—Steel and concrete across direct bands at center—After 8 days the steel stress remained very nearly constant throughout the year. The concrete deformations were relatively small also and in general similar to Group 6 and 7 above but on a reduced scale. The general behavior of Groups 3 and 10 was like that of Groups 1 and 6.

Group 4 (Fig. 6) and Group 8 (Fig. 8).—Steel and concrete at center of span of diagonal bands—During the first six months the steel stress increased and thereafter remained practically constant. The compressive deformation in the concrete increased very little during the first 74 days of falling temperature but with the turn in the temperature curve increased much faster during the next six months. In the final four months of nearly constant temperature the increase was very small.

Group 5 (Fig. 6) and Group 9 (Fig. 8).—Steel and concrete at center of

span of direct bands—These groups show much the same behavior as groups 4 and 8 above, but with much smaller deformations in the steel.

COMPARISON WITH FULLER & MORE TESTS.

Figs. 3 to 10 are not plotted to any consistent scale so far as time is concerned (the time is stated but the intervals between readings are not to scale), and Fig. 11 has therefore been plotted to enable a comparison to be made between the data of Fuller & More's tests and that presented herewith. In this figure "a" represents the data of a cylinder loaded to 1150 lb. per sq. in. constant load, "b" a cylinder loaded to 650 lb. per sq. in. constant load, and "c" the compressive deformation at the panel center in the Schwinn Building. All three are plotted to the same scales. The striking agreement in the increase in deformation for corresponding periods of the tests is shown by the following comparison:

Curve.	Period Considered.	Unit Deformation.		Ratio.
		Beginning.	End.	
a	1 to 230 days	0.00053	0.00280	1 : 5.28
c	1 to 230 days	0.00010	0.00052	1 : 5.20
b	1 to 55 days	0.00040	0.00105	1 : 2.62
c	1 to 55 days	0.00010	0.00024	1 : 2.40

GENERAL CONCLUSIONS.

This is the only test of its kind that I know of, and conclusions should not be stated in any very general terms. Conditions were exceptionally favorable, however, both as to the test building and as to recurrence of the same temperature at the beginning and end of the test. Only one panel was loaded and other loadings may give somewhat different results. The readings taken during the year were limited to those gage lines which showed maximum stresses in the 24-hour test and the results given here are maxima rather than averages for the various groups. With the conditions of the test in mind certain conclusions seem warranted.

1. The stresses in the steel show practically no change during the last four months of the test (249 to 379 days).

2. The concrete deformations show progressive increase during the year. The average increase of the five groups was much more marked in the first five months (1 to 165 days) than in the last seven. This increase in the average compressive deformation is in accord with experiments on cylinders under laboratory conditions.

3. There occurred a shifting of a portion of the moment during the first six months of the test, stresses adjoining the columns decreasing while those at the center increased. This action must be considered in establishing design coefficients.

4. The increase in the total moment, negative and positive, figured from the steel stresses, was relatively small. While not subject to exact calculation, this increase appears to have been consideraby less than 10 per cent for the year, with no increase in the last four months.

5. Comparison of the data of this test with that secured by Professors Fuller & More, leads to the conclusions that the actual maximum compressive stresses at the center of the panel was not in excess of 350 lb. per sq. in. at the full test load.

6. Both concrete and steel stresses at the end of the year appear to be materially less than those to be expected under the Chicago Ruling governing the design of flat slabs.

7. This test, taken together with other time tests of concrete in compression, would indicate that the reported values of the compression in the concrete in previous extensometer tests of flat slab buildings are somewhat in excess of the true stresses actually present in those tests. This applies especially to the final readings when the full load has been in place for 24 hours.

8. Further tests of this nature would seem to the writer to promise a material addition to our understanding of the deportment of reinforced-concrete structures under load. Such tests should serve to reduce the factor of ignorance and make possible more economical design in those materials. The "straight-line theory" of reinforced concrete is clearly inapplicable to this test even with the low stresses present in the concrete and steel.

TEST OF A FLAT CONCRETE TILE DOME REINFORCED CIRCUMFERENTIALLY.

By W. A. Slater* and C. R. Clark.*

The test described in this paper was made on a dome-shaped structure (Fig. 1) which in plan formed a panel 10 ft. wide and 12 ft. long between the interior surfaces of the supporting walls. Fig. 2 gives a view of the completed panel. The structure consisted of a thin shell made of concrete tiles (Fig. 15) $2\frac{1}{4}$ in. thick supported on concrete walls 8 in. thick, the entire structure being leveled up by means of a 1 : 3 : 8 mix of cinder concrete 2 in. thick at the center and increasing to 6 in. at the ends. The average thickness of the cinder concrete fill was about $4\frac{1}{2}$ in. The concrete shell was reinforced by means of circumferential rods $\frac{3}{8}$ in. in diameter spaced 9 in. on centers.

The concrete shell formed a portion of a hollow sphere 100 ft. in diameter. Thus any vertical section passing through the center of the dome formed the arc of a circle of 100 ft. diameter. This gave a rise of 3 in. between the inner surface of the wall and the center of the dome on the 10-ft. span and of 4.1 in. on the 12-ft. span. The shell was built of tiles of the form shown in Fig. 3. The tiles were made from a mortar of 1 part of cement to 2 parts of sand. They were laid in a mortar made up of 1 part cement, 1 part of lime, and 6 parts of sand. They were laid in concentric circular courses, beginning at the corners and working toward the center without the aid of forms as shown in Fig. 4. The corners were filled in with partial courses until a complete circular course could be laid. Each tile dovetailed into a tile of the previous course in such a way as to support the individual tiles by cantilever action until the course was completed.

The tiles were laid by a man inexperienced in this work but no difficulty was encountered in placing them so that they kept their place and gave a creditable appearance to the completed structure. The true spherical form of the dome was maintained by the use of a template which consisted of an arm cut to conform to the arc of a true circle whose diameter coincided with the vertical axis of the dome and whose radius was 50 ft. As this arc rotated about the vertical axis an arc of a true spherical dome was generated. By laying the tiles to conform to the position indicated by this template a spherical dome was secured. Each course formed a zone which after completion was reinforced with a circular rod of $\frac{3}{8}$-in. round steel placed in the circumferential groove formed by the notches cast in the tiles. Mortar similar to that above described was slushed into the groove around the reinforcement to give a bearing to the reinforcement.

After a course was completed it depended no longer upon cantilever action in the tile for support. Although cantilever action was still present to some extent any yielding of the course would bring compression into the

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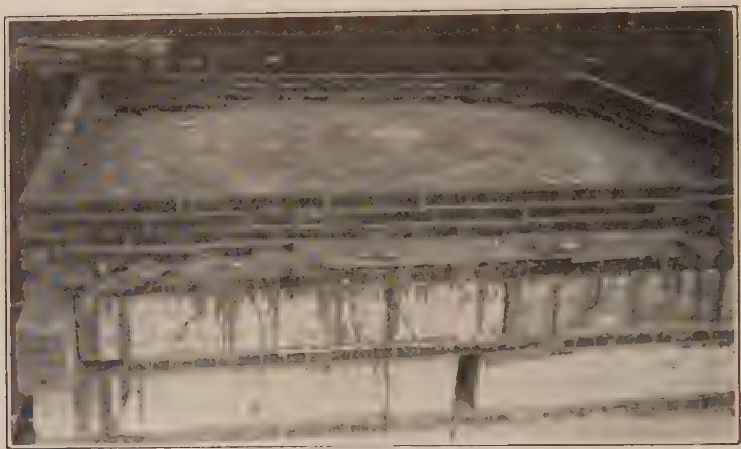

FIG. 1.—VIEW OF DOME BEFORE CINDER CONCRETE FILL HAD BEEN PLACED

FIG. 2.—VIEW OF COMPLETED STRUCTURE DURING IMPACT TESTS.

upper edges of the tile in a circumferential direction and this compression would be resisted by the tension in the circumferential reinforcement and by friction against the adjacent outer course. The reinforcing rods for the incomplete courses in the corners were carried along the exterior edge of the

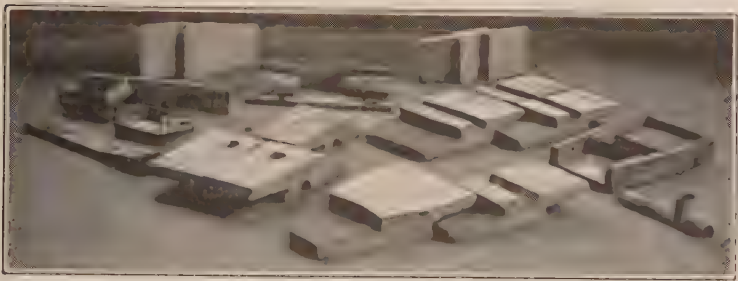

FIG. 3.—VIEW SHOWING FORM OF TILES USED.

dome on the supporting wall as indicated by dotted lines in Fig. 6, and were thoroughly encased in mortar for anchorage.

Lack of promptness in taking these precautions to anchor the bars caused the premature destruction of the dome the first time its construction was attempted. The outer courses of the structure had been completed without

FIG. 4.—VIEW OF DOME WHEN PARTIALLY COMPLETED.

first grouting the reinforcing bars of the outer incomplete courses. When all but two or three courses of tiles had been laid the reinforcing bars for courses eight and nine were seen to loosen throughout most of their position on the side wall, Fig. 6, and let down the entire construction rapidly but not instantaneously. With the second attempt greater attention was paid to the rein-

forcing and anchoring of each course as soon as it was completed and no difficulty of the kind above named was experienced. Where it was necessary to splice bars a lap of about 50 diameters was provided, and the bars were hooked at the ends as may be seen in Fig. 4. Measurements of deformation were taken at one of these splices to see how effective the splices were. (See gage lines 70–71, Fig. 6.)

The dome was tested with static load at somewhat less than 30 days after erection. The load consisted of pig-iron (Fig. 5), which was placed in increments of 2000 lb. ($16\frac{2}{3}$ lb. per sq. ft.) until a load of 8000 lb. had been reached and in increments of 4000 lb. from then until the maximum load of 16,000 lb. had been reached. The pig-iron was weighed to insure that the correct amount of load was applied. Observations taken during this vertical load test were, (*a*) strain-gage readings at gage lines shown in Fig. 6; (*b*) deflection

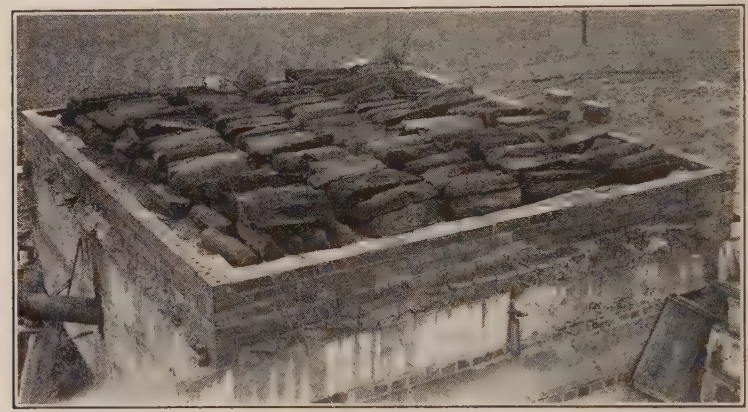

FIG. 5.—VIEW OF DOME WITH 16,000-LB. LOAD IN PLACE.

readings taken at the center and at the deflection points shown in Fig. 6; and (*c*) outward deflection of the walls at the centers of the four sides. The latter measurement was to determine whether the walls were called upon to resist thrust. Fearing the possibility of sudden failure of the thin shell of the dome thus letting the entire load fall through with serious consequences to the observers, use was made of the brick pier shown in Fig. 15 and the walls, to support a framework in the northeast corner of the panel. This was intended to serve as a shelter in case of unexpected failure of the dome.

After the load of 133 lb. per sq. ft. (total load 16,000 lb.) had been applied the entire load was removed. After another series of no-load readings had been taken loads of 38 and 100 lb. per sq. ft. were applied to the north half and strain-gage readings were taken after each load to determine the effect of the eccentric load.

After the static load tests had been completed an impact test was made by dropping a stone various distances and observing the effect on the structure.

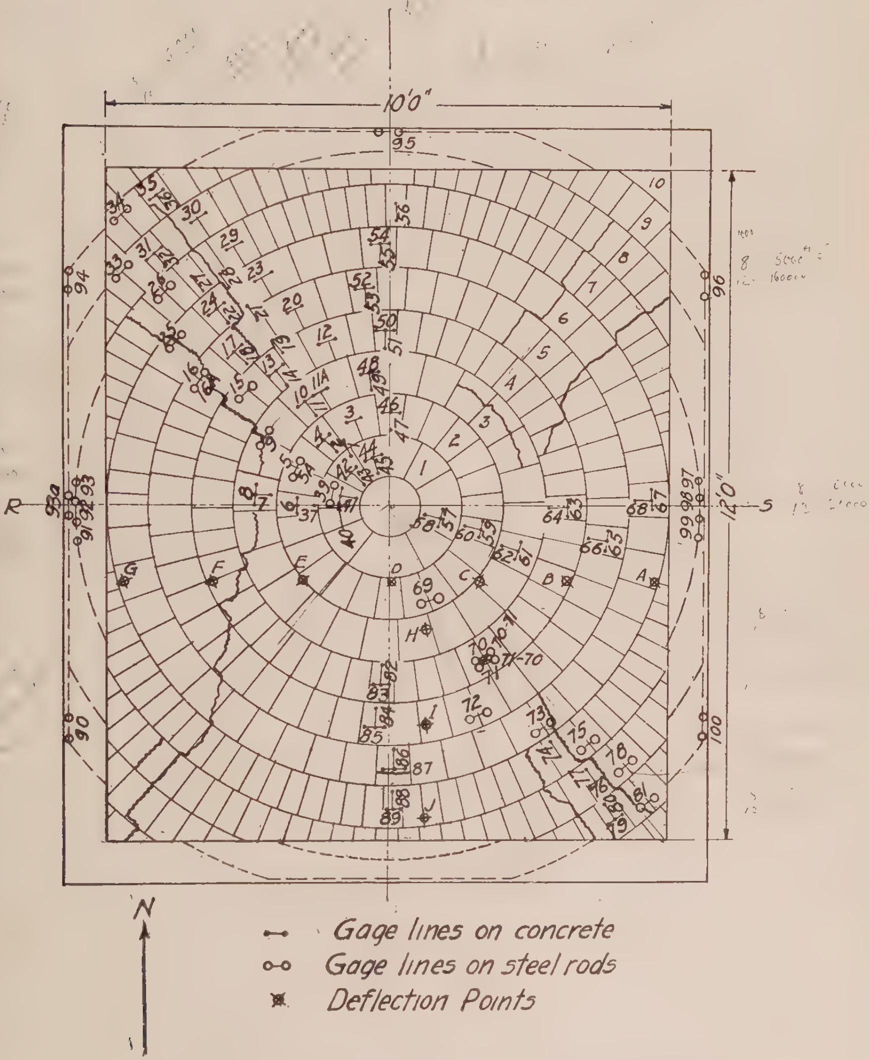

FIG. 6.—PLAN SHOWING LOCATION OF GAGE LINES, DEFLECTIONS, POINTS, AND CRACKS.

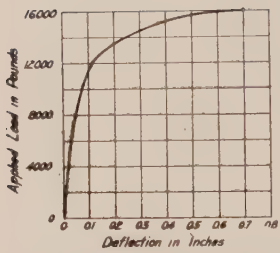

FIG. 7.—VERTICAL DEFLECTION AT POINT D UNDER STATIC LOAD.

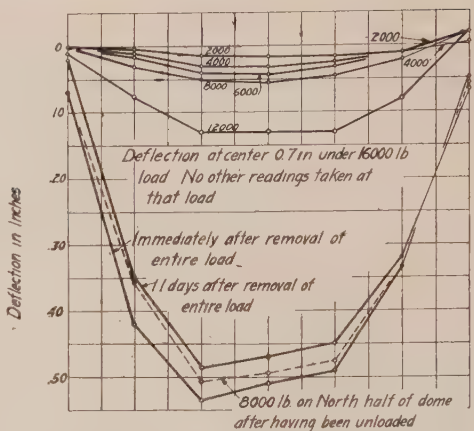

FIG. 8.—VERTICAL DEFLECTION ON EAST-WEST CROSS-SECTION FOR VARIOUS LOADS.

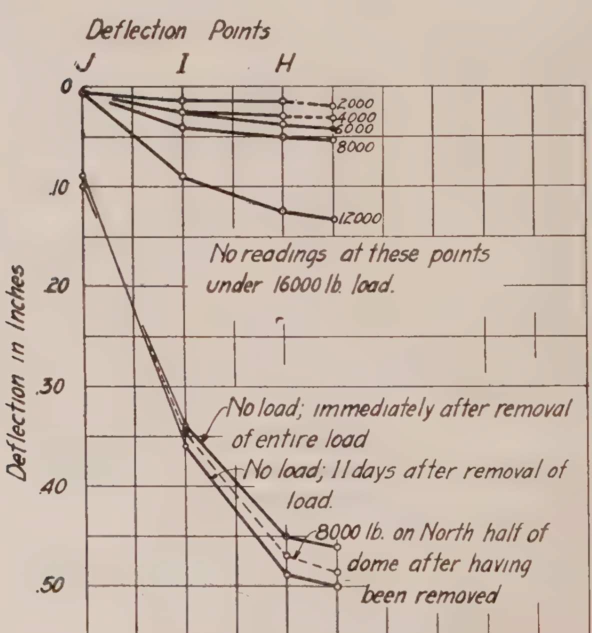

FIG. 9.—DEFLECTION ON NORTH-SOUTH CROSS-SECTION FOR VARIOUS LOADS.

After the dome had been removed a determination was made of the ability of the walls to resist thrust by applying to them known horizontal loads and measuring the amount of horizontal deflection of the wall under these loads.

Figs. 7, 8 and 9 show the observed deflections under static load. At the maximum load of 16,000 lb. only one deflection reading, that at deflection

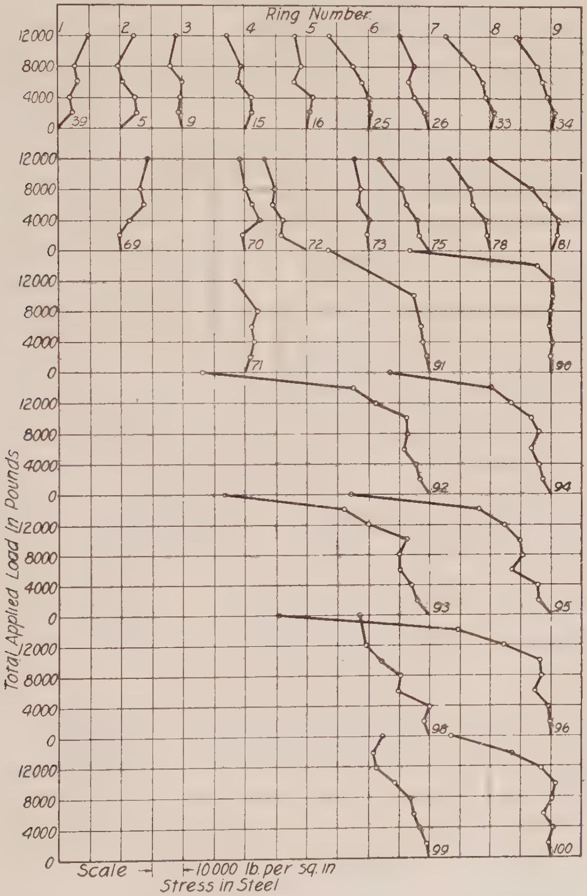

FIG. 10.—LOAD-DEFORMATION DIAGRAMS FOR CIRCUMFERENTIAL GAGE LINES ON REINFORCEMENT.

point *D*, was taken. At this load some of the stresses had gone beyond the yield-point and it was considered unsafe to go under the structure for the purpose of getting the complete set of readings. The lever arm deflection apparatus shown in Fig. 15 could be read from the outside, however, and this indicated a deflection of about 0.65 in. After the load had been in place

long enough to make it appear that danger was not imminent a more accurate deflection reading (that on point D) was ventured. This showed 0.7 in. deflection. Fig. 7 shows the load-deflection curve for point D. This, as well as the strain-gage readings, shows that at the last load the dome had yielded.

It will be noted in Fig. 8, which gives deflections on a cross-section of the dome 15 in. south of the center, that the central portion of the dome went down nearly uniformly, that is, at all stages of the test a line through points representing the deflections at E, D, and C would be nearly straight, indicating that the cracks which appear in Fig. 6 allowed the portion of the dome between these cracks to settle integrally, throwing the main support out toward the corners of the supporting walls. In some of the strain-gage data there are other indications of the same action, but there is rather too much uncertainty about these indications to place much reliance upon them. Immediately after the removal of the load, deflection readings were taken and

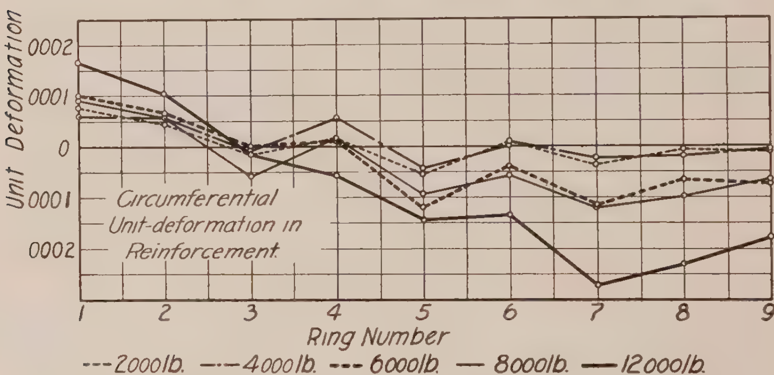

FIG. 11.—DISTRIBUTION OF CIRCUMFERENTIAL UNIT-DEFORMATIONS IN REINFORCEMENT FOR VARIOUS LOADS.

these showed a recovery of 27 per cent of the maximum deflection under full load. After standing 11 days without load another 6 per cent recovery had taken place, giving in all 33 per cent recovery. In order to see what effect an eccentric static load would have on the structure a load of 6000 lb. was uniformly distributed over the north half of the dome and readings of deflection and deformation were taken. As shown in Fig. 8 by the dotted line the effect was only slight.

In Fig. 9 deflections on a north-south section, 8 in. from the north-south center line are plotted and these indicate that the eccentric load did not change the shape of the north-south section more than that of the east-west section. The effect of the eccentric load seems to have been well distributed and not more marked than that of the same amount of load distributed uniformly over the entire area in the first application of load.

Figs. 10 to 14 show the unit-deformations computed from the readings taken with the strain gage. As explained in connection with the deflection

readings it was considered unsafe to remain under the dome when the 16,000-lb. load was in place and consequently at this load strain-gage readings were obtained on only those points which were accessible from outside the structure.

Fig. 10 gives the distribution of the unit-deformations found in a circumferential direction in the reinforcement. At the low loads the deformations were small, and the distribution of deformation is somewhat erratic. At the 12,000-lb. load the deformations were beginning to increase more rapidly and

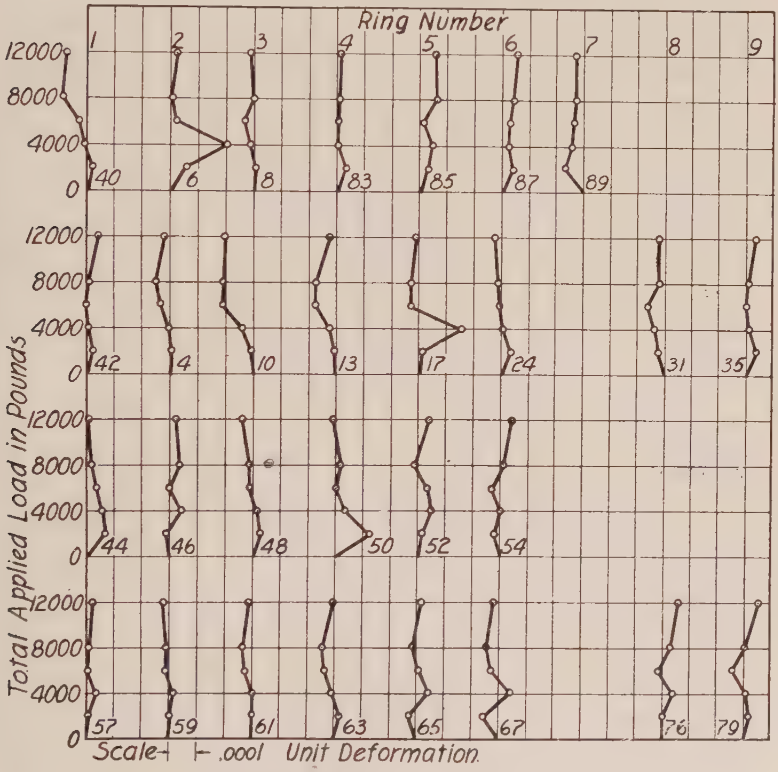

FIG. 12.—LOAD-DEFORMATION DIAGRAMS FOR CIRCUMFERENTIAL GAGE LINES ON CONCRETE.

Fig. 11, which shows the average unit-deformation for the various rings, indicates compression in the central portion of the dome in a circumferential direction, changing progressively into tension in the outer portion. This distribution curve indicates a point of zero stress about 30 to 33 in. from the center. It is statically necessary that the sum of the tensions occurring circumferentially across a radial section normal to a side of the structure be equal to the sum of the compressions across the same radius. It is apparent,

however, that in this figure the area between the tension curve and the axis is greater than the area between the compression curve and the axis. It is evident that in order to obtain the total stresses there should be added to the areas here shown the areas corresponding to whatever stresses were carried by the concrete in compression and in tension. In compression the concrete would be fully effective and a considerable area would be added to the compression area shown in the figure, which gives only the steel stresses. In

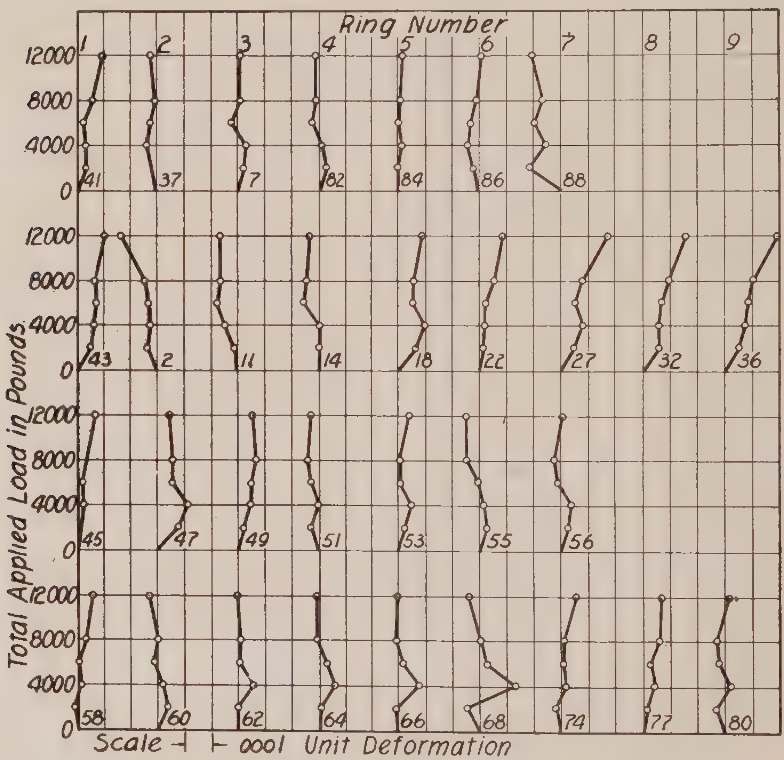

FIG. 13.—LOAD-DEFORMATION DIAGRAMS FOR RADIAL GAGE LINES ON CONCRETE.

tension the concrete was cracked and it may be that very little should be added to the outer or tension area. The effectiveness of the concrete in compression and its ineffectiveness in tension may account for the inequality between the tension area and the compression area as shown.

Fig. 12 shows the deformations measured circumferentially in the concrete on the under surface of the shell of the dome. There is so much irregularity in these curves that little can be made from them. This may be due partly to temperature variations within the structure, partly to different

deformations across mortar joints from the deformations at other points, and partly to changes caused by formation of cracks as the test proceeded. It is known that there was a considerable amount of variation of temperature within the enclosure of the structure due to artificial heat at time of taking observations. This heat could not be distributed with entire uniformity and it would affect the surface of the concrete much more than it would the steel which was embedded rather deeply in the concrete. That there was, at least in some cases, a difference between deformations in gage lines which crossed joints from those which did not cross joints, may be seen from a comparison of gage line 23 (Fig. 14) with gage line 24 (Fig. 12). These gage lines are located in similar positions and close together, yet they show very large differences in deformation. The largeness of the deformation which may be

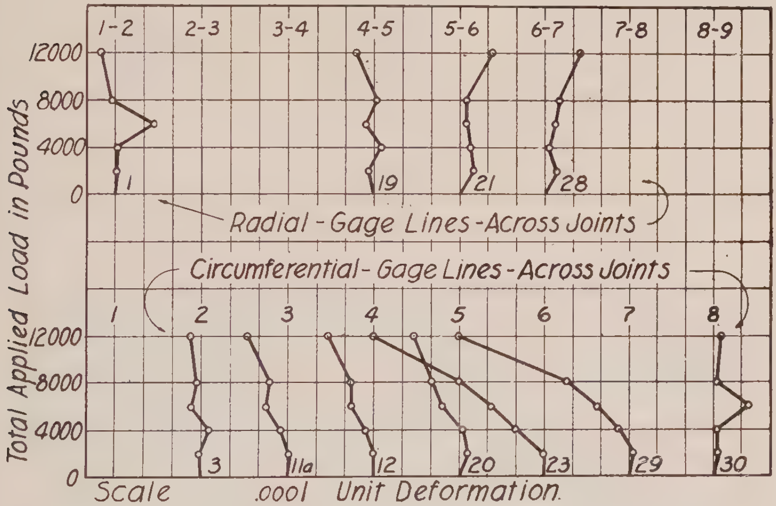

FIG. 14.—LOAD-DEFORMATION DIAGRAMS FOR RADIAL AND CIRCUMFERENTIAL GAGE LINES ACROSS MORTAR JOINTS.

found generally in circumferential gage lines which cross joints in the tension region helps to confirm the suggestion previously made that the concrete in tension was not fully effective, and that there probably was some separation of tiles at mortar joints even where no cracks could be detected.

The measured radial deformations are shown in Fig. 13. These were all measured on concrete and the same sources of difficulty in interpretation are present as in the case of circumferential deformations measured on the concrete surface except that since compression in a radial direction should be expected generally, the effect of the joints should be less marked.

With the thin shell as the only structural support and with no radial reinforcement, danger of sudden failure and of peculiar susceptibility to impact was considered. As a preliminary test a 50-lb. weight was dropped from a

height of about $6\frac{1}{2}$ ft. about a dozen times at the center of the dome where the thickness of the cinder concrete fill was only 2 in. This caused a slight vibration of the structure but had no permanent effect. Accordingly a more severe impact test was devised. A stone weighing 195 lb. (about 10 in. wide and 27 in. long) was raised varying distances and dropped on the dome.

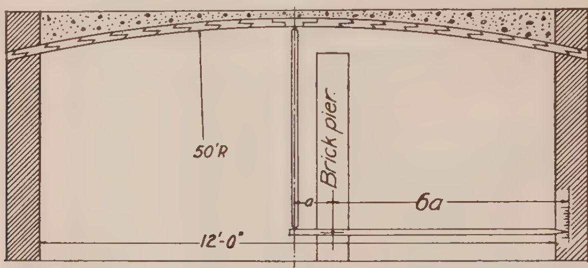

FIG. 15.—CROSS-SECTION OF STRUCTURE SHOWING ARRANGEMENT OF DEFLECTION APPARATUS.

The derrick improvised for raising the stone to the desired height is shown in Fig. 2. The hook with which the stone was attached to the block and tackle used in this apparatus was provided with a trip so that the stone could be suddenly released and allowed to fall entirely free from all restraint.

FIG. 16.—VIEW SHOWING FRACTURE DUE TO IMPACT AT CENTER OF DOME.

The first seven blows were applied at the center of the dome where the thickness of the cinder concrete fill was 2 in. In order to approximate the conditions under which such a structure would have to meet service tests 1-in. boards were at first laid on the top of the dome to receive the force of the blow. It was soon ascertained that the effect was slight enough that in order to produce

appreciable permanent effects the boards would need to be removed and they were not used after the application of the first two blows. To determine the effect of each blow the multiplying lever apparatus sketched in Fig. 15 was used to determine the deflection at the center at the instant of impact. Due

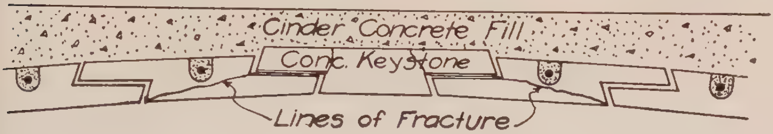

FIG. 17.—SKETCH SHOWING MANNER OF FAILURE AT CENTER UNDER IMPACT.

to the energy imparted to the pointer by the impact, the indicated deflection may have been greater than the actual deflection but it does not seem likely that it could have been less. The heights of the drop varied from 4 ft. to 6 ft. 8 in.

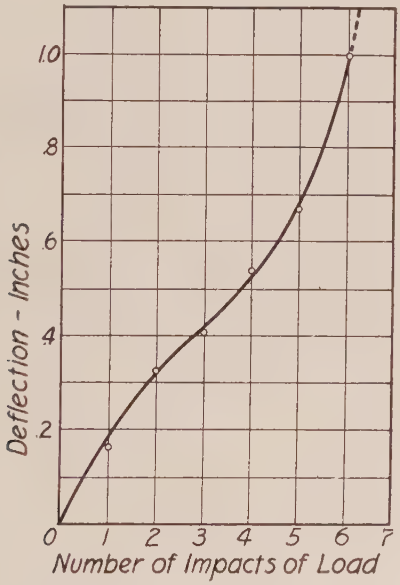

FIG. 18.—DEFLECTION AT CENTER UNDER IMPACT.

The data of the test are given in Table I. It will be seen that no puncturing of the surface took place until the seventh blow. The hole caused at this time was roughly oval with axes of about 13 and 15 in. respectively. This hole may be seen in Fig. 2 at the center of the structure. Also a view from the interior looking upward toward the center of the dome is given in Fig. 16.

The latter photograph shows something of the form of the fracture. The first failure, as indicated in remarks in Table I, caused a rupture around the keystone of the dome as shown in Fig. 17 and a shearing (or tension) along a portion of the first course of tile outside the keystone. The position of the latter surface of failure indicates that the compression in the radial direction due to the impact was practically in line with the axis of the tile.

TABLE I.—DATA OF IMPACT TEST NO. 1.

Drop No.	Height Dropped.	Deflection, in.	Remarks.
1	4 ft. 0 in.	0.16±	
2	6 " 6 "	0.33±	
3	6 " 8 "	0.41±	
4	6 " 8 "	0.55±	A few radial cracks at center, and edges of keystone chipped.
5	6 " 8 "	0.67±	Edges of keystone broken off.
6	6 " 8 "	1.0	Appearance of diagonal tension starting about 15 in. from center.
7	6 " 8 "	....	Punched small hole in center.

The deflections at the center of the dome due to the blows are shown in Fig. 18. It should be recognized that the observations were taken instantaneously and may be somewhat crude but it is believed that they represent roughly the amount of deflection.

The above described test showed an unexpected resistance to impact. To determine whether the resistance to impact would be less if the blows were applied eccentrically the derrick shown in Fig. 2 was moved to a point 3 ft. south of the center and a similar test was made. In this instance the height of the drop was uniformly 6 ft. 8 in. and no board was placed on the

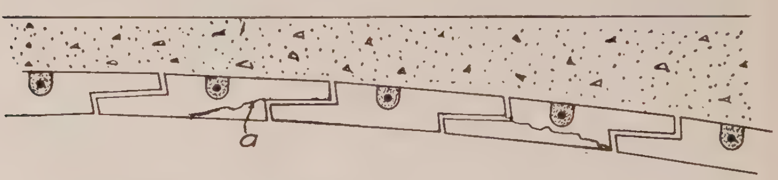

FIG. 19.—FORM OF FRACTURE DUE TO IMPACT 3 FT. FROM CENTER.

cinder concrete. The twelfth blow caused a hole somewhat larger than the one caused at the center but still not large enough to let the stone fall through. The form of the fracture was about as shown in Fig. 19 in cross-section. It would seem that the occurrence of the crack at *a* Fig. 19 indicates that the lateral thrust was well distributed even in the case of eccentrically applied impact loads.

The stability of the structure as a whole did not seem to be impaired by the tests previously described and the derrick was then moved to a point near the north end of the slab and an excavation made through the cinder concrete deep enough to expose the upper surface of the tiles. This area was approx-

imately a rectangle 14 in. by 34 in. The 195-lb. stone was dropped a distance of 7 ft. 1 in. upon the unprotected dome. The first blow cracked the dome but made no hole. Fig. 20 shows the effect of this blow as photographed from beneath the structure. One more blow from a height of 7 ft. 1 in. was applied at this point. This made a small hole but the stone did not fall through. The derrick was then moved to a point about $1\frac{1}{2}$ ft. south of the center, that is, to a point about half way between the first two points of application. Here the stone was raised to a height of 6 ft. 8 in. and dropped upon the cinder fill which was $2\frac{1}{4}$ in. thick at this point. At the seventh blow a hole was made and the stone fell through but did not take out all the material between holes marked 2 and 1. (See Fig. 21.) The stone was then dropped at the point marked 5 and the concrete was broken out from 1 to 4 but the structure still showed considerable resilience.

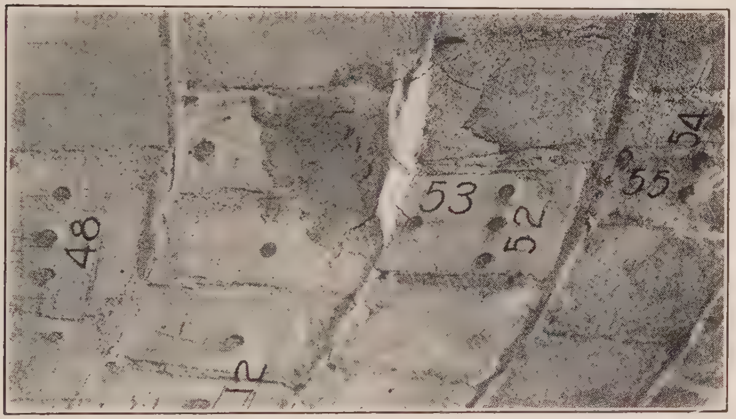

FIG. 20.—VIEW SHOWING FRACTURE DUE TO IMPACT ON UNPROTECTED SURFACE.

In wrecking the structure the central portion of the shell was broken out from end to end. At that time there was still sufficient strength to support the weight of two men jumping upon the shell on one side of the center. This fact is mentioned merely as indicating the considerable degree of toughness in the structure.

In all the tests previously described the failures were entirely local in nature and the thing that had been feared—that due to the absence of radial reinforcement a fracture at one point might drag down the entire ring and all that portion of the structure within the ring—was not realized.

The method of measuring the outward deflection of the side walls was crude and the deflection probably could not be read with certainty with an error of less than 0.01 in. The observed deflection on one side at the 8000-lb. (67 lb. per sq. ft.) load was 0.02 in. No deflection of the end walls could be

detected. Readings of deflection of walls were not recorded at higher loads. With this allowance for error the maximum deflection becomes 0.03 in. In order to find out how much lateral thrust was required to produce this lateral deflection of the walls a test was made after the dome had been wrecked and removed from the supporting walls. In this test a horizontal load was applied at the middle of the side walls close to the top of the wall. The outward deflection due to the known horizontal load was measured and has been plotted in Fig. 22. In the static test of the dome whatever horizontal thrust came upon the walls was not applied at a single point as in the test under consideration but was distributed along the length of the wall. Consequently the load required to cause a given deflection of the wall would be greater for the static

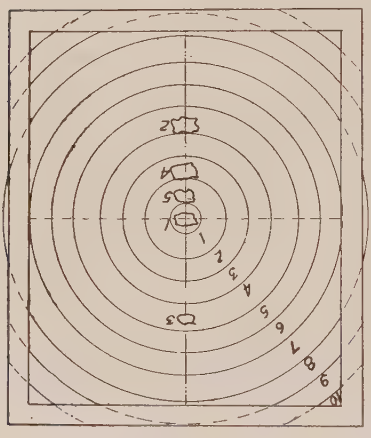

FIG. 21.—SKETCH SHOWING LOCATION OF HOLES DUE TO IMPACT.

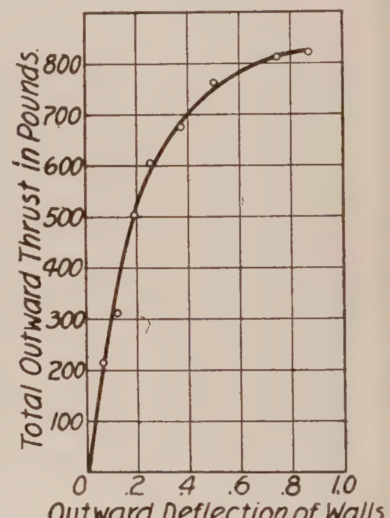

FIG. 22.—HORIZONTAL DEFLECTION OF WALLS UNDER KNOWN HORIZONTAL LOAD.

test of the dome than for the test of the wall in which the load was applied at a single point at the center of the length of the wall. If the deflection due to the concentrated load were twice as great as the former the horizontal thrust due to the vertical load of 8000 lb. on the dome would be not more than 100 lb. as indicated by Fig. 22. This, of course, is negligible. There is no indication that for the higher loads the lateral deflections of the walls increased more rapidly than the load increased. It seems certain that the resistance afforded by the walls to horizontal thrusts was negligible.

The test of the walls by application of horizontal load indicates that even if the action of the dome required considerable elongation of the span there was not sufficient stability to the walls to resist this increase in the span

length. On the other hand the small deflection of the non-resistant supporting wall under the action of the vertical load upon the dome indicates that even if supporting walls of such a structure are capable of resisting considerable horizontal thrust only a very small amount of horizontal deflection (or lateral movement of the support) is necessary to relieve the thrust to be resisted, and to relieve the walls of the necessity of carrying the thrust.

The idea of a dome constructed in concentric courses of tiles in which each course is supported during its construction by the adjacent course without the use of forms and in which each course, after its completion, is reinforced with a steel ring, was conceived by Mr. Clark. The dome was designed and built by him and tested by the writers jointly. Parts of the test data were embodied in a thesis by Mr. Clark for the degree of Master of Architecture at the University of Illinois.

On the basis of the data recorded in this paper an attempt was made to analyze the action of a flat dome of this type. It seems that there are certain laws of action disclosed and yet there are inconsistencies which prevent a great deal of dependence on conclusions, and no analysis is presented. In a general way, however, the test has afforded data from which it is believed the action of such a structure may be predicted to a certain extent. The test has indicated that a structure built after the manner of the dome tested is feasible and exhibits considerable toughness. For cases in which a large unobstructed space with a domical ceiling capable of carrying a load on the floor above is desired it seems that there are possibilities in such construction as that described. In any contemplated use of such a structure it is obvious that a circular plan would be more advantageous than a square plan, and that a square plan would be more advantageous than a rectangular plan.

THE INFLUENCE OF TOTAL WIDTH ON THE EFFECTIVE WIDTH OF REINFORCED-CONCRETE SLABS SUBJECTED TO CENTRAL CONCENTRATED LOADING.

By A. T. Goldbeck.*

About five years ago the United States Office of Public Roads increased the scope of its activities by adding a bridge department to its organization. Standard designs were prepared and useful data were tabulated relative to bridge construction. But it was very forcibly brought to the attention of those engaged on this work that many problems yet remained in bridge design which were solvable only through the collection of experimental data. One of these perplexing problems was to decide on a reliable assumption for the distribution of stress in reinforced-concrete slabs carrying concentrated wheel loads. No published information could be found on this subject and the laboratory of the office started a series of tests to obtain results of value to the designing engineer. In the intervening period of five years since the tests were begun, the laboratory has collected much information on stress distribution, some of which has been published.†

It has been shown by the tests that the stress distributed itself over a much greater width of slab than originally was thought probable. In general it may be said that when a concentrated load is placed in the center of a reinforced-concrete slab, supported only at two ends, the stress produced is distributed over a width equal to approximately twice the span length. The stress immediately under the load is greatest, and at a distance equal to the span length, measured on each side of the load, it becomes very small or vanishes to zero. Deductions were drawn from the previously published tests regarding the design of slabs whose width exceeded twice the span length. From the beginning it was thought desirable to make use of the already existing theory of rectangular beam design, and the experimental results obtained were therefore analyzed to determine the "effective width" of the slab, that is the width having the same resisting moment as the entire width of slab under stress, when the maximum stress in the center of the slab is considered constant over this width.

As pointed out in the previous papers, the effective width varies from about 0.7 of the span length to a little more than the span length, and in

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* Testing Engineer, U. S. Office of Public Roads and Rural Engineering.

† "Tests of Reinforced-Concrete Slabs under Concentrated Loads" by A. T. Goldbeck, 1913 *Proceedings* of the American Society for Testing Materials.

"Tests of a Large-Sized Reinforced-Concrete Slab," by E. B. McCormick, 1915 *Proceedings* of American Concrete Institute.

"Further Tests of Large-Sized Reinforced-Concrete Slabs," by A. T. Goldbeck and E. B. Smith, 1916 *Proceedings* of American Concrete Institute.

"Tests of Large-Sized Reinforced-Concrete Slabs," by A. T. Goldbeck and E. B. Smith, *Journal of Agricultural Research*, May 8, 1916.

designing a slab supported at its two ends and having a width equal to twice the span length or more, a reasonable assumption for the effective width would be 0.7 of the span length. The formulas for rectangular beams could then be applied to the design of slabs by substituting $0.7L$ for b. Such an assumption was an approximation based on the experimental data and was apparently safe in all cases tested.

In many slabs used in construction, however, the width is less than twice the span length, and may vary all the way from a narrow rectangular beam through various widths up to that in which the width is even more than double the span. It is the object of this paper to present test results showing how the effective width of a reinforced-concrete slab depends on the total width of the slab when it is supported at two ends only, and is subjected to a central concentrated load.

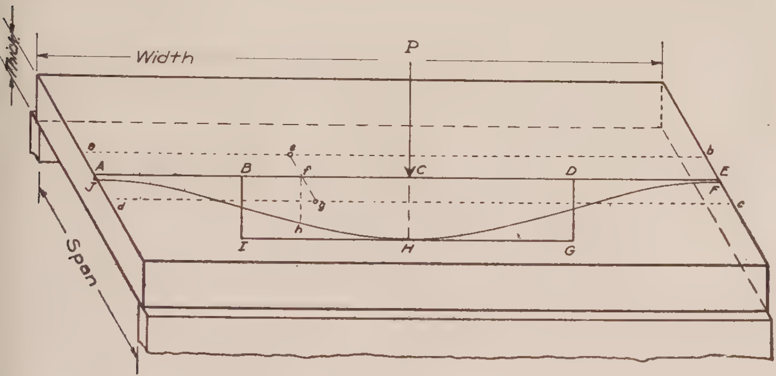

FIG. 1.—DIAGRAM SHOWING ASSUMED CONDITIONS IN SLAB UNDER LOAD.

DESCRIPTION OF SPECIMENS.

No. 835. Made June 26, 1914, tested July 17 to Nov. 16, 1914. 1 : 2 : 4 gravel, 16-ft. span, 32 ft. wide, total depth 12 in., effective depth $10\frac{1}{2}$ in. Reinforcing $\frac{3}{4}$ in. square, $1\frac{1}{2}$ in. above bottom. Spacing, 10.5 in. = 0.75 per cent.

No. 930. Made March 10, 1915, tested March 29 to June, 1915. 1 : 2 : 4 gravel, 16-ft. span, 32 ft. wide, total depth 10 in., effective depth $8\frac{1}{2}$ in. Reinforcing $\frac{3}{4}$ in. square, $1\frac{1}{2}$ in. above bottom. Spacing, 8.87 in. = 0.75 per cent.

No. 934. Made Oct. 2, 1915, tested Nov. 2, 1915, to Jan. 5, 1916. 1 : 2 : 4 gravel, 16-ft. span, 32 ft. wide, total depth 7 in., effective depth 6 in. Reinforcing $\frac{1}{2}$ in. square, 1 in. above bottom. Spacing, 5.56 in. = 0.75 per cent.

No. A19. Made Jan. 14, 1916, tested Feb. 15 to March 7, 1916. 1 : 2 : 4 gravel, 6-ft. span, 12 ft. wide, total depth 7 in., effective depth 6 in. Reinforcing $\frac{1}{2}$ in. square, 1 in. above bottom. Spacing, 5.55 in. = 0.75 per cent.

No. A20. Made Jan. 28, 1916, tested March 10 to 20, 1916. 1 : 2 : 4 gravel, 6-ft. span, 12 ft. wide, total depth 4 in., effective depth 3 in. Reinforcing $\frac{3}{8}$ in. plain square, 1 in. above bottom. Spacing, 0.75 per cent = 6.26 in.

No. A21. Made April 12, 1916, tested May 24 to 25, 1916. 1 : 2 : 4 gravel, 6-ft. span, 12 ft. wide, total depth 5 in., effective depth 4 in.

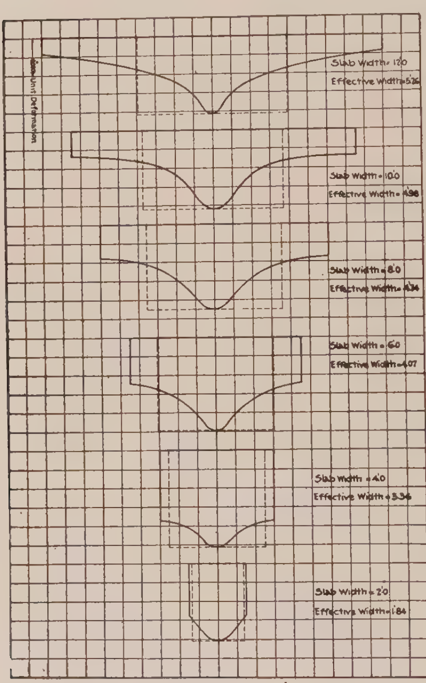

FIG. 2.—CONCRETE DEFORMATIONS. SLAB A-19.

Span 6 ft. Effective Depth 6 in. Steel 0.75 per cent.

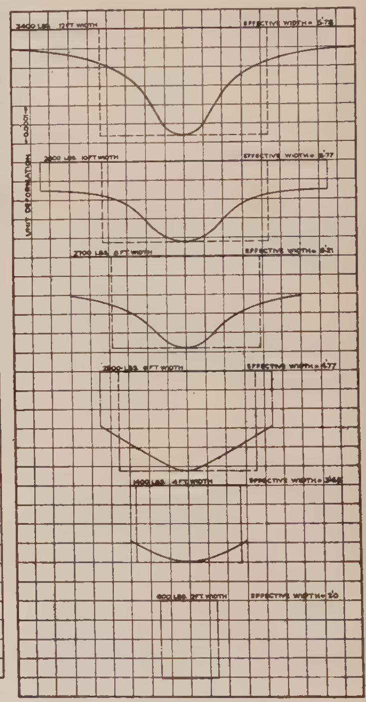

FIG. 3.—CONCRETE DEFORMATIONS. SLAB A-20.

Span 6 ft. Effective Depth 3 in. Steel 0.75 per cent.

Reinforcing $\frac{1}{2}$ in. plain square, 1 in. above bottom. Spacing, 8.333 in. = 0.75 per cent.

No. A22. Made May 18, 1916, tested June 29 to July 18, 1916. 1 : 2 : 4 gravel, 16-ft. span, 32 ft. wide, total depth 16 in., effective depth 15 in. Reinforcing $\frac{3}{4}$ in. plain square, 1 in. above bottom. Spacing, 5 in. = 0.75 per cent.

No. A23. Made July 31, 1916, tested Sept. 12 to 23, 1916. 1 : 2 : 4 ravel 6-ft. span, 12 ft. wide, total depth 6 in., effective depth 5 in.

Reinforcing ½ in. plain square, 1 in. above bottom. Spacing, 6.66 in. = 0.75 per cent.

No transverse reinforcement was used in any of the above described slabs.

METHOD OF TESTING.

The slabs of 6-ft. span were mounted on I-beam supports placed across one of the extension arms of a 200,000 lb. general testing machine, and the loads were applied over an 8-in. bearing area by means of the machine. Extensometer readings were made with a 20-in. Berry strain-gage on both the

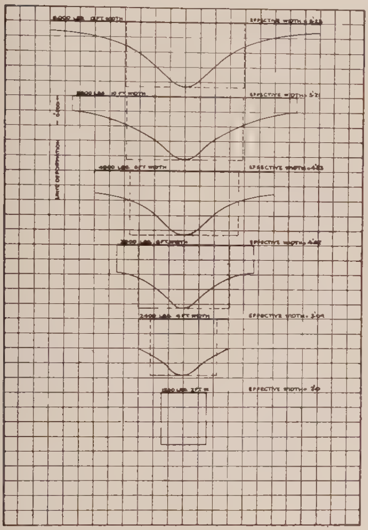

FIG. 4.—CONCRETE DEFORMATIONS. SLAB A-21. Span 6 ft. Effective Depth 4 in. Steel 0.75 per cent.

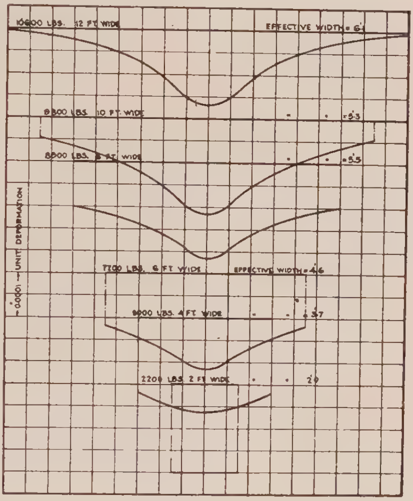

FIG. 5.—CONCRETE DEFORMATIONS. SLAB A-23. Span 6 ft. Effective Depth 5 in. Steel 0.75 per cent.

steel and the concrete. Deflection measurements were likewise taken by means of a special apparatus designed by the author for measuring the wear of concrete roads.* By means of this apparatus readings could be taken to the nearest 0.001 in. The largest slabs of 16-ft. span were placed on reinforced-concrete supports and loads were applied with a hydraulic jack. Load readings were obtained by observing with the aid of an Ames dial the deflections of a calibrated chrome-nickel steel beam, and strain-gage, and deflection readings were taken in a manner similar in principle to that used in the case of the small slabs. The details of the loading apparatus have been described

* "An Apparatus for Measuring the Wear of Concrete Roads," by A. T. Goldbeck, *Journal of Agricultural Research*, Feb. 14, 1916.

in the previously published papers and will, therefore, not be treated in detail here.

After the slabs were cast and while they were still soft, rows of bolts were set in their top surfaces which, when withdrawn a few days later, left holes for the insertion of the plugs and feathers to be used for splitting off sections of the slabs as the tests progressed. In this way one specimen was made to serve for a number of tests with various widths of slab. When the slab had been tested having its full width, sections were split off the ends and the width was thus decreased. Another test was then made with this decreased width, and end sections were again split off until finally a very narrow width of slab resulted. This procedure of testing the same slab cut to different widths

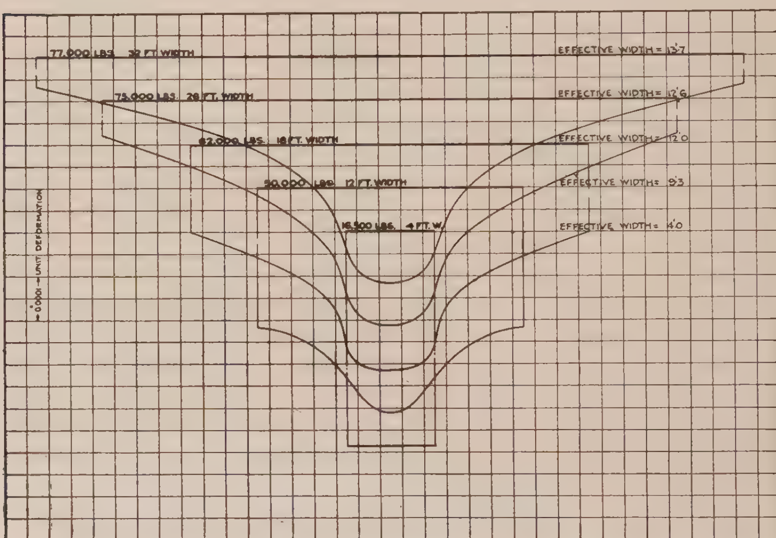

FIG. 6.—CONCRETE DEFORMATIONS. SLAB A-22.
Span 16 ft. Effective Depth 15 in. Steel 0.75 per cent.

could be pursued without affecting the results, because the load applied at no time stressed the specimen beyond its working stresses.

As already stated, the objective of the tests was to obtain results that would lead to the rational design of a reinforced-concrete slab of any width when supported at its two ends, and when subjected to a central concentrated load. The dangerous section of a slab under such conditions, is the middle section, and the method of procedure in testing was to measure the deformations in the concrete and in the steel over the middle section, parallel to the supports. The deflections of the slab along its center were measured throughout its width. The results of the deformation readings in the steel and in

the concrete were finally used to obtain the "effective width" of the slab, and the method of applying the test results for finding this value may be described as follows:

EFFECTIVE WIDTH.

The width of the slab that should be used in the rectangular-beam formulas when applied to slab design has been termed the "effective width" of the slab. It is that width over which, if the stress were constant and equal to the maximum stress under actual conditions, the resisting moment would equal the

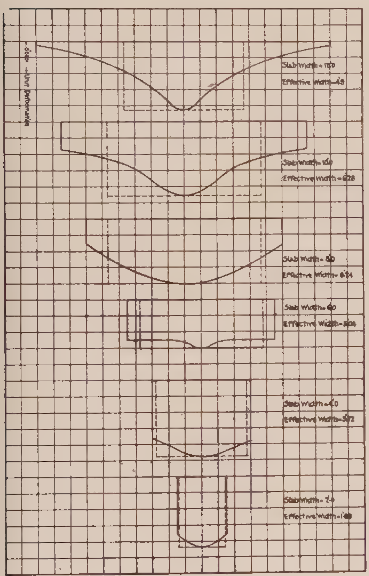

FIG. 7.—STEEL DEFORMATIONS. SLAB A-19.

Span 6 ft. Effective Depth 6 in. Steel 0.75 per cent.

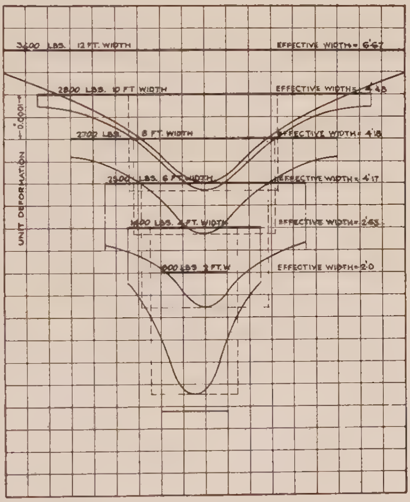

FIG. 8.—STEEL DEFORMATIONS. SLAB A-20.

Span 6 ft. Effective Depth 3 in. Steel 0.75 per cent.

resisting moment of a slab of the same depth and full width, but having varying stress distribution. If the straight-line theory of stress distribution from neutral axis to upper fibers is assumed to be applicable to slabs, the resisting moment of a given slab is dependent on the total stress in the concrete or steel at the dangerous section. The total stress in the concrete, however, is governed by the stresses in the top fibers, and these stresses are proportional to the unit deformations. If, then, there are two slabs of equal depth, one having uniform distribution of deformations and the other a varying distribution, but with their maximum deformations identical, they will likewise

have equal resisting moments if the summations of the deformations over their respective widths are identical.

In Fig. 1, which represents a slab in position on two supports with a concentrated load P, is illustrated the method of obtaining "effective width." Strain-gage readings are taken of the fiber deformations perpendicular to the supports, as indicated at *eg*. These concrete deformation values are plotted to scale, as, for instance, at *fh*, giving the deformation curve *JHF*, inclosing the area *AJHFE*. This curve shows the variation of stress from the center to each of the two free edges of the slab, and the area under the curve is a

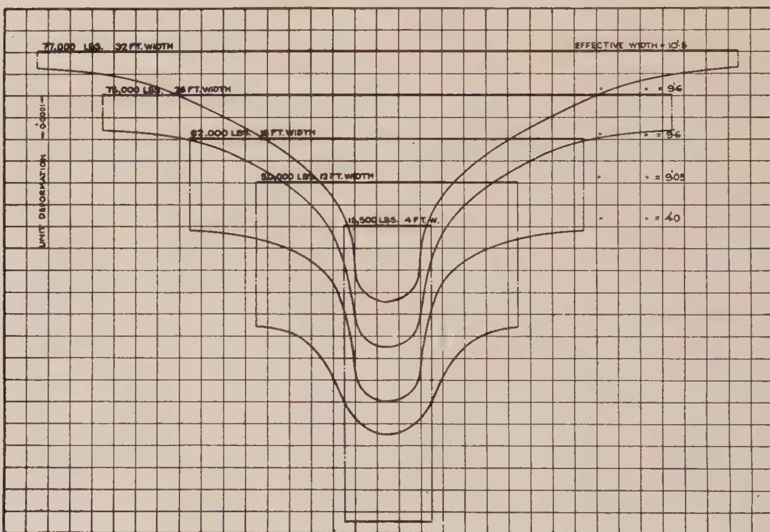

FIG. 9.—STEEL DEFORMATIONS. SLAB A-22.
Span 16 ft. Effective Depth 15 in. Steel 0.75 per cent.

function of the total concrete-resisting moment of the slab. The area *BDGI*, obtained by dividing the area *AJHFE* by its maximum ordinate *CH*, has the same total concrete-resisting moment with the stress uniformly distributed as the whole slab, and its width *BD* is that which may be effective in furnishing sufficient resistance under these conditions to carry the load. The width *BD*, obtained in this manner, is the "effective width."

RESULTS OF TESTS.

The results of the tests made on slabs numbered 835, 930 and 934 are not presented in detail in this paper as they have already been published elsewhere.* The deformations in the concrete and in the steel of the remain-

* *Journal of Agricultural Research*, May 8, 1916.

ing slabs, however, are given in full in Figs. 2–9. The deflections are likewise shown in Figs. 10–12.

Referring to the curves of deformation of both the concrete and the steel, it will be noted that the maximum deformation exists under the load, and becomes smaller at the sides, but even at a distance equal to the span length measured from the center, there is still some deformation and consequently some stress. It was found that readings could be taken in the concrete much easier and with more assurance of their accuracy than in the steel. The steel readings, moreover, were influenced by the restraining effect of the concrete. In the vicinity of the cracks greater elongation takes place than

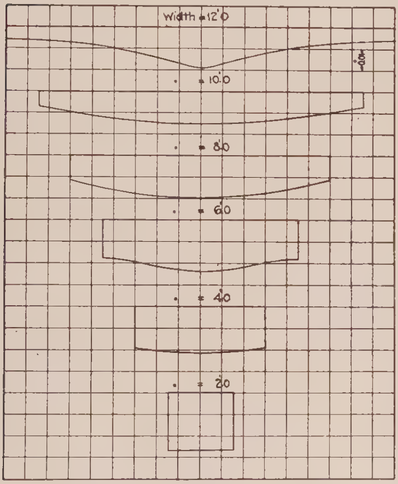

FIG. 10.—DEFLECTIONS. SLAB A-19.

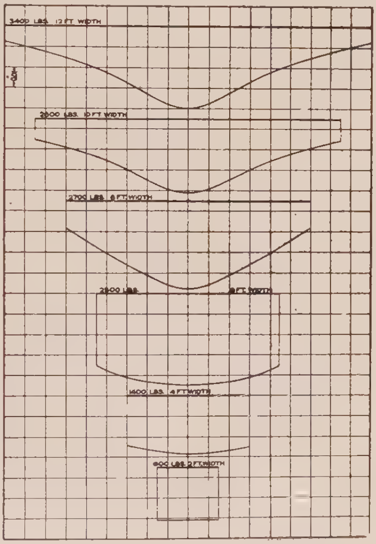

FIG. 11.—DEFLECTIONS. SLAB A-20.

where the concrete remains intact and the total deformation in the gage length is no indication of the maximum unit deformation in the steel. The steel readings were, therefore, not regarded as being quite so reliable as the concrete readings. It will be well to point out that the gradual "flow"* effect of concrete, even when under small stress, renders it impossible to translate the deformations measured into unit fiber stresses. However, by taking account of the permanent set in calculating the unit deformations, the curves of deformation are fairly representative of the stress distribution.

Having plotted the curves of unit deformation, their areas were obtained with the aid of a polar planimeter. These areas divided by their maximum

* "Flow of Concrete Under Sustained Loads," by E. B. Smith, 1916 and 1917 *Proceedings* of American Concrete Institute.

ordinates equal the effective widths of the slabs tested, and the values of the effective widths thus obtained are shown on the respective curves.

CONCLUSIONS FROM INVESTIGATIONS.

The final results of the investigation are best shown on Fig. 13 where the ratios of the total width to span are plotted as abscissas and the ratios of effective width to span as ordinates. As the width of slab increases the ratio of effective width to span length shows considerable variation. This variation, however, is representative of what might be expected in actual structures, and apparently does not follow any law so far as thickness is concerned. In view of this variation in results, it was thought desirable to interpret them

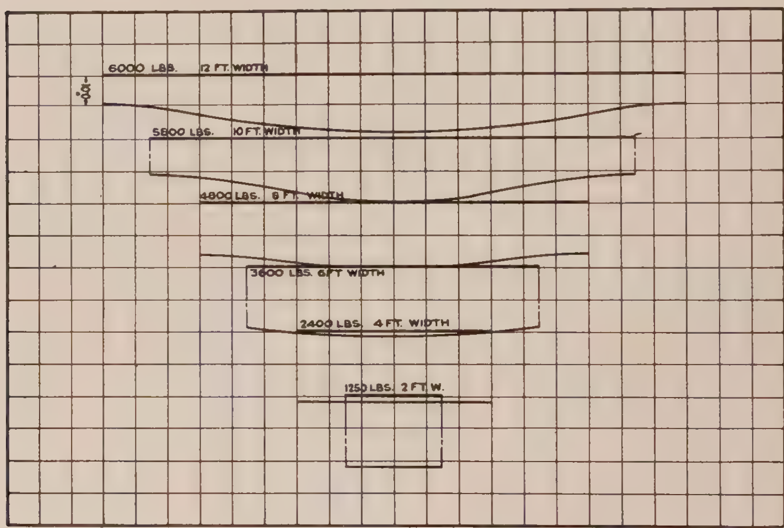

FIG. 12.—DEFLECTIONS. SLAB A-21.

conservatively, and the curve drawn through the lower edge of the belt of points probably best gives the desired relation of effective width and total width. This relation is likewise expressed in the following table:

$\frac{\text{Total Width}}{\text{Span}}$	$\frac{\text{Effective Width}}{\text{Span}}$
0.1	0.1
0.2	0.2
0.3	0.28
0.4	0.37
0.5	0.44
0.6	0.5

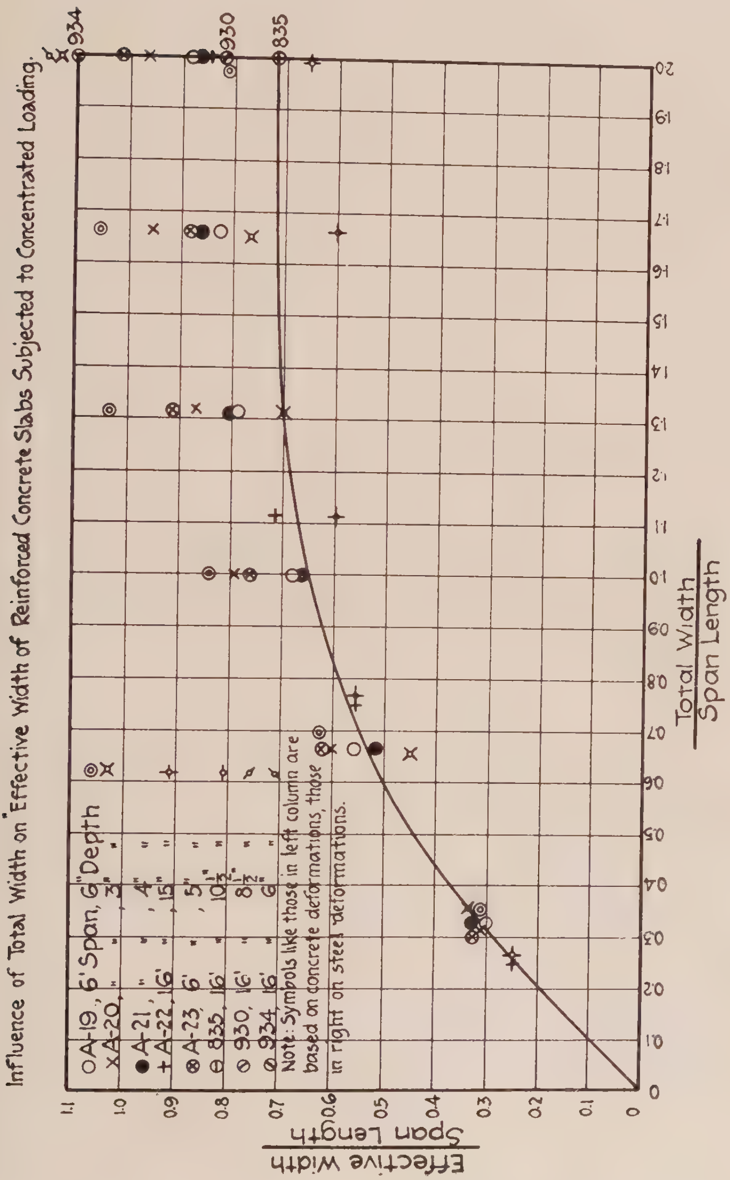

FIG. 13.—INFLUENCE OF TOTAL WIDTH ON "EFFECTIVE" WIDTH OF REINFORCED CONCRETE SLABS SUBJECTED TO CONCENTRATED LOADS.

$\frac{\text{Total Width}}{\text{Span}}$	$\frac{\text{Effective Width}}{\text{Span}}$
0.7	0.55
0.8	0.58
0.9	0.62
1.0	0.65
1.1	0.67
1.2	0.68
1.3	0.70
1.4	0.71
1.5	0.72
1.6	0.72
1.7	0.72
1.8	0.72
1.9	0.72
2.0	0.72

The above values can be used for spans up to 16 ft. and probably for longer spans, although no longer spans were tested.

DESIGN OF SLAB OF ANY WIDTH.

The design of a slab of any width can be accomplished by using the formulas for narrow rectangular beams and substituting for the breadth (b) the value obtained from the above table. Thus, suppose a slab of 12-ft. span and 15-ft. width is to be designed to carry a central concentrated load of 20,000 lb. then

$$\frac{\text{Total Width}}{\text{Span}} = \frac{15}{12} = 1.25$$

From the table, the effective width $= 0.68 \times 12$ ft. $= 8.15$ ft. Consider the entire load of 20,000 lb. to be carried by a width of 8.15 ft. and use the ordinary formulas for rectangular-beam design.

In conclusion the author wishes to acknowledge the assistance of E. B. Smith, who personally made many of the tests reported and who also performed the calculations necessary for their graphical presentation.

FIELD TESTS OF CONCRETE FOR THE NEW YORK SUBWAYS.

By J. G. Steinle.*

At the last meeting of the American Concrete Institute Mr. Robert Ridgway, Engineer of Subway Construction for the New York Public Service Commission, presented a paper entitled "Concreting New York's Subways." In that paper he described the character of the materials employed, the methods of inspection, and the manner of mixing and placing concrete on the work.

The purpose of the present paper is to offer additional facts and details concerning the inspection and testing of the concrete and concrete aggregates.

Field Tests.

Aside from testing the cement and concrete aggregates, tests are regularly made on concrete as it is being placed. Samples are taken direct from the forms and placed in metal cylinders (8 in. diameter by 16 in. high) and tested, three at 28 days and three at 90 days. These samples are taken from the forms rather than from the mixer, it being desirable to test the concrete as it actually enters the structure and not during any part of the process of handling. In cases where concrete is delivered from a central mixing plant, part of the water is added at the site, and a fair sample can be obtained only from the forms. Any effects due to delays in delivering concrete, or any effects of handling would also be detected by sampling in this manner. Our experience has shown that more uniform field test specimens are obtained from the forms than from the mixer, especially in the cases of concrete of very wet consistency.

Storage of Specimens.

Inasmuch as the method of curing a test specimen has a considerable effect on its strength, it is stored under conditions approximating the concrete in the structure. The specimens are left in the molds and stored on the work for two days and then delivered to the laboratory, where the molds are removed and the concrete cylinders stored in a damp cellar. The specimens are sprinkled daily, and two days before testing are allowed to dry out in the laboratory, as it has been found that more uniform results are obtained by testing them dry. During the winter months it is often difficult to store specimens on the work, as they freeze more easily than mass concrete; frozen specimens usually developing only one-third of the strength developed by unfrozen specimens.

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* Assistant Engineer, Public Service Commission, New York City.

METHOD OF TESTING.

In testing a sample of concrete it is essential that the pressure be distributed uniformly over the ends on which the force is applied, and to accomplish this all specimens which have uneven ends are capped with plaster of paris. In ordinary cases, however, it has been found that by cushioning the ends of the cylinders with "Compo" board, or a similar material, a uniform bearing can be obtained, as the "Compo" board will yield to any small projections. Cylinders tested in this way will not ordinarily break in the head, or split longitudinally, on account of concentrated pressure. In order to provide for specimens in which the planes of the ends are not parallel, the compression head on the testing machine is equipped with a special ball-and-socket joint.

FORMS.

The metal forms used in the making of the test cylinders may be of special interest, as they can be procured at a very low cost. Ordinary 8-in. water pipe is cut into 16-in. sections with parallel ends and a longitudinal slot is made extending the length of the cylinder; the cylinder can then be clamped together by two metal bands fitted with a nut and bolt. It was found that the split pipe will spring apart sufficiently to allow it to be removed from the concrete cylinder when the pressure of the bands is released.

EFFECT OF CONSISTENCY.

The result of testing a large number of concrete cylinders made in the field seems to indicate that, next to the cement, the most important factors affecting the strength of the concrete are the field conditions under which it is made. Of these factors the consistency of the mix perhaps causes the greatest variations in the strength, especially on specimens tested up to 28 days. In spite of the fact that the test cylinders show that concrete of wet consistency possesses a low compressive strength, it is at times necessary on subway work to use wet mixture on account of the difficulty of passing it through chutes, and because of complicated forms and reinforcements. The effect of an excess of water is to swell the volume and reduce the density. In an excessively wet mixture there will be more or less separation of materials, and concrete of uniform quality will not ordinarily be produced. An excess of water will also interfere with the chemical reaction of the cement and cause the concrete to become both slow setting and slow hardening. Field tests show that a concrete of a quaking consistency will give the best results.

COMPARISON OF FIELD TESTS WITH CONCRETE IN THE STRUCTURE.

In order to establish the relation between the strength of the test specimens and mass concrete and to determine how close the factor of safety possessed by the actual structure approximated the one assumed in the design, a series of tests were made on concrete cut from the finished work. Before making these tests, the places from which these samples were to be cut were selected and later 6 x 6 x 12-in. specimens were made from the

concrete as it was being placed in the forms at these points. One-half of the test specimens were stored in the moist room in the usual way, and the remaining were stored in wet sand in the subway near the points from which the mass concrete specimens were to be cut. Sixty days after the concrete was placed, sections were cut from the structure, which were then roughly dressed, and ground to exact dimensions, stored in the moist room and tested at 90 days. Tables 1 and 2 show the results obtained:

TABLE 1.—COMPARISON OF FIELD TESTS WITH CONCRETE IN THE STRUCTURE.

Size of Specimens—6 x 6 x 12 in., 1 : 2 : 4 Concrete 90 Days Old—Strength in lb. per sq. in.

Cut from 12-in. Wall.	Same Batch Poured in Molds and Stored on the Work.	Same Batch Poured in Molds and Stored in Moist Room.	Consistency.	Fine Aggregate.	Coarse Aggregate.
4 Av. 3095	4 Av. 2880	4 Av. 2870	Wet	L. I. beach sand	L. I. beach gravel
4 Av. 2410	....	4 Av. 1720	Very wet	L. I. washed bank sand	L. I. washed bank gravel
4 Av. 2415	2 Av. 2585	3 Av. 2060	Wet	L. I. beach sand	L. I. beach gravel
4 Av. 2760	4 Av. 2020	4 Av. 1775	Very wet	L. I. beach sand	L. I. beach gravel
16 Av. 2670	10 Av. 2480	15 Av. 2110			

Same Materials Mixed in Laboratory and Stored on the Work.	Same Materials Mixed in Laboratory and Stored in Moist Room.	Consistency.	Fine Aggregate.	Coarse Aggregate.
4 Av. 2135	4 Av. 2080	Wet	L. I. beach sand	L. I. beach gravel
4 Av. 2225	4 Av. 1930	Wet	L. I. washed bank sand	L. I. washed bank gravel
4 Av. 1980	4 Av. 1705	Wet	L. I. beach sand	L. I. beach gravel
4 Av. 1900	4 Av. 1910	Wet	L. I. beach sand	L. I. beach gravel
16 Av. 2060	16 Av. 1910			

TABLE 2.—TESTS MADE ON OTHER SPECIMENS CUT FROM WORK.

Age.	Size of Specimen.	No. of Specimen.	Proportion.	Strength, lb. per sq. in.	Cut from	Remarks.
13 yr. 6 mo.	6 x 6 x 12	2	1 ; $2\frac{1}{2}$: 5	4095	Roof arch, existing subway	
11 yr. 2 mo.	6 x 6 x 12	1	1 : $2\frac{1}{2}$: 5	3020	Existing subway	
3 yr. 3 mo.	6 x 6 x 12	3	1 : $2\frac{1}{2}$: $4\frac{1}{2}$	2896	Existing subway	
2 yr. 6 mo.	6 x 6 x 12	3	1 : $2\frac{1}{2}$: $4\frac{1}{2}$	3342	Existing subway	
1 yr. 8 mo.	6 x 6 x 12	4	1 : 2 : 4	3070	New subway	
1 yr. 7 mo.	6 x 6 x 6	1	1 : 2 : 4	1910	New subway	Small gravel
42 days	6 x 6 x 12	1	1 : 2 : 4	2600	Tunnel	Concrete placed by pneumatic blower
35 days	6 x 6 x 12	3	1 : 2 : 4	3120	Underpinning	Concrete deposited through 8-in. pipe with a vertical fall of 40 in. and after leaving pipe it had a clear fall through the air of 20 ft.

These tests indicate that the concrete in the structure will show at the end of 90 days a compressive strength 25 per cent greater than concrete cast in molds and tested in the usual way.

It will also be observed that whereas a concrete of very wet consistency when sampled and tested in the usual way will show low results, the *same concrete* when cut from the structure will not show the same relative weakness. This may be partially explained by the fact that a large part of the water and lighter particles, in actual practice, rise to the top of the form and are not confined in the mass of the concrete.

TABLE 3.—EFFECT OF TEMPERATURE ON TIME OF SET.

Time of Set Tests.

Sample No. 1....	In laboratory air 66° F...........	Initial set	1 hr. 50 min.	Final	5 hr. 00 min.
Sample No. 2....	In laboratory air 66° F...........	Initial set	2 hr. 20 min.	Final	5 hr. 30 min.
Sample No. 3....	In laboratory air 68° F...........	Initial set	1 hr. 48 min.	Final	4 hr. 20 min.
Sample No. 1....	In air over ice water 44° F........	Initial set	5 hr. 45 min.	Final	Lost
Sample No. 2....	In air over ice water 44° F........	Initial set	6 hr. 45 min.	Final	Lost
Sample No. 3....	In air over ice water 45° F........	Initial set	4 hr. 0 min.	Final	8 hr. 28 min.
Sample No. 1....	In water 62° F..................	Initial set	Over 7 hr.	Final	Lost
Sample No. 2....	In water 62° F..................	Initial set	Over 7 hr.	Final	Lost
Sample No. 3....	In water 70° F..................	Initial set	3 hr. 55 min.	Final	7 hr. 55 min.
Sample No. 1....	In ice water 38° F...............	Initial set	40 hr. 0 min.	Final	Lost
Sample No. 2....	In ice water 38° F...............	Initial set	40 hr. 0 min.	Final	Lost
Sample No. 3....	In ice water 37° F...............	Initial set	21 hr. 30 min.	Final	36 hr. 0 min.

Tensile Tests, lb. per sq. in.

	Sample No. 1.		Sample No. 2.		Sample No. 3.
	7 Days.	28 Days.	7 Days.	28 Days.	7 Days.
In ice water 38° F. average................	130	340	145	365	88
Over ice water in air 44° F................	282	428	292	400	207
In tanks water 62° F......................	240	363	250	365	243

It should also be borne in mind that the concrete in the actual structure is under a considerable hydrostatic pressure which may increase its density, although these tests have failed to show any definite relation existing between the strength of the concrete and the head of the concrete above it.

Inasmuch as all these tests were of the same material, their weight should be an indicator of their density, but our laboratory tests show that no relation exists between the weight and density or between the weight and compressive strength of the specimen. It can be readily seen that a specimen of wet concrete may contain an excess of stone or gravel and be very heavy; nevertheless, it may have a low density and a low compressive strength.

CEMENT.

It has been frequently found that cement which passes standard specifications and which will produce normal results at laboratory temperature,

may vary somewhat on the work. Some cement will set more slowly than others in cold weather, while others will set much more quickly in hot weather when mixed under field condition. We have found, for instance, that the action of gypsum in retarding the setting is not always permanent, but may disappear during storage of the cement. Cement shipped in summer may be slow setting when leaving the mill, but on arrival at the work, the extreme heat in the cars may cause it to become quick-setting. The effect of low temperature on the time of set and strength is illustrated by Table 3.

In making up the "time of set specimens" and briquettes, those to be stored in cold air, or cold water, were made up of previously cooled ingredients, and the briquettes stored in ice water were allowed to harden in air at 44° F. for 24 hours.

RETEMPERING CONCRETE.

Whereas no retempered concrete is allowed on the work and its use is prohibited, tests were made to determine how seriously the strength of concrete is affected by retempering, and the following five series of tests were made on cylinders 8 in. diameter by 16 in. high. The results are shown in Table 4.

TABLE 4.—FIELD TESTS OF REGAGED CONCRETE.

Size of specimen 8 in. diameter by 16 in. high. 1 : 2 : 4 portland cement concrete. Tested at 28 days.

A denotes specimens made as soon as batch was mixed (machine).
B denotes specimens made from same batch and regaged one hour after mixing.
C denotes specimens made from same batch and regaged again two hours after mixing.
D denotes specimens made from same batch and regaged again three hours after mixing.
E denotes specimens made as soon as batch was mixed (hand).
F denotes specimens made from same batch and regaged one hour after mixing.
G denotes specimens made from same batch and regaged two hours after mixing.
H denotes specimens made from same batch and regaged three hours after mixing.

Test No.	Lb. per sq. in.				Consistency.	Fine Aggregate.	Coarse Aggregate.
	A	*B*	*C*	*D*			
1	4 Av. 1402	4 Av. 1342	4 Av. 1444	4 Av. 1117	Wet	Washed bank sand	Washed bank gravel
2	3 Av. 690*	4 Av. 1948	4 Av. 1968	4 Av. 1421	Very wet	Excavated sand	Beach grave
3	4 Av. 1409	4 Av. 1404	4 Av. 1673	4 Av. 1382	Wet	Washed bank sand	Trap rock
4	4 Av. 2985†	4 Av. 2900	3 Av. 2535	4 Av. 2718	Wet	Beach sand	Trap rock
‡	12 Av. 1932	16 Av. 1899	15 Av. 1860	16 Av. 1660			
	E	*F*	*G*	*H*			
5	4 Av. 1732	3 Av. 1567	4 Av. 1668	4 Av. 1597	Wet	Beach sand	Beach gravel

* This set of specimens frozen, result not considered in final average.
† This set could not be broken at the capacity of the machine (153,000 lb.).
‡ General average.
NOTE.—Sets 1 to 4, inclusive, were made in the field. Set 5 made in the laboratory (hand mixed).

Another series of tests were made, using the cement separately: 135 oz. of portland cement was gaged with 23 per cent of water for 5 min., and allowed to stand for 1½ hr., at which time initial set had taken place; it was then thoroughly remixed with a small addition of water to bring it to a normal consistency and 9 briquettes made and tested with the following results:

	24 HOURS. LB. PER SQ. IN.	7 DAYS. LB. PER SQ. IN.	28 DAYS. LB. PER SQ. IN.
	368	569	714
	312	647	685
	311	620	695
Average....	330	612	698

The balance of this mortar, after a period of 2½ hr. longer, and after initial set had taken place the second time, was regaged to a normal consistency with a small addition of water, and gave the following results:

	24 HOURS. LB. PER SQ. IN.	7 DAYS. LB. PER SQ. IN.	28 DAYS. LB. PER SQ. IN.
	288	530	687
	275	630	650
	306	610	695
Average....	290	590	677

After remaining undisturbed 1 hr. longer, or 5 hr. from first gaging, and initial set having taken place a third time, it was regaged to normal consistency with the following results:

	24 HOURS. LB. PER SQ. IN.	7 DAYS. LB. PER SQ. IN.	28 DAYS. LB. PER SQ. IN.
	210	560	643
	248	540	630
	270	568	575
Average....	243	556	616

From the same cement 90 more ounces were taken and gaged with 23 per cent of water continuously for 1½ hr., when 9 briquettes were made and tested giving the following results:

	24 HOURS. LB. PER SQ. IN.	7 DAYS. LB. PER SQ. IN.	28 DAYS. LB. PER SQ. IN.
	434	618	774
	350	727	716
	343	744	738
Average....	376	696	743

The balance of the mortar, gaged 1 hr. longer, or 2½ hr. from the beginning, was tested as follows:

	24 HOURS. LB. PER SQ. IN.	7 DAYS. LB. PER SQ. IN.	28 DAYS. LB. PER SQ. IN.
	373	624	729
	336	680	748
	350	638	690
Average....	353	647	722

While 45 oz. gaged with 23 per cent of water and made up immediately into 9 briquettes, gave the following:

	24 HOURS. LB. PER SQ. IN.	7 DAYS. LB. PER SQ. IN.	28 DAYS. LB. PER SQ. IN.
	408	674	787
	372	707	670
	350	625	770
Average....	377	669	742

The "initial set" on the cement used was about 1½ hr., and the final or hard set, 4¼ hr.

It may be of interest to note in the mortar tests that the batch which was mixed continuously for 1½ hr., gave results identical with those gaged only for five minutes.

MIXING.

The effect of the time of mixing of concrete on its compressive strength cannot be well established from data obtained from our field tests, as most of our specimens are taken from the forms and not directly from the mixer. The total mixing of the concrete is made up in part by that received in the mixer and in part by passing through chutes, rehandling, etc. The actual effect of the treatment in the mixer on the strength of the specimen made in this way is partially lost. The following tests, made on concrete mixed for 1 min. 30 sec. and 10 sec., and then passed through long chutes, will illustrate this point:

TEST NO.	TIME MIXED.	AGE.	CONSISTENCY.	ULT. STRENGTH. LB. PER SQ. IN.
1	10 sec.	7 days	Wet	1268
2	30 "	7 "	"	996
3	60 "	7 "	"	1144

The best and most uniform test results have been obtained on concrete mixed for at least one minute after all the ingredients had been added. Thorough mixing will remove films of air adhering to particles and will reduce the amount of air confined in the mix; it will also increase the fluidity of the mix, thereby reducing the amount of water required. Test specimens, which show

low compressive strength, usually fail because of the breaking of the bond between the mortar and the coarse aggregate. The certainty of securing a bond between the broken stone or gravel and the mortar is greatly increased by continued mechanical mixing, as a film of silt or a coating of stone dust enveloping the larger particles is removed and a cleaned surface is exposed to the mortar. This could not be easily accomplished by hand-mixing and may account for the prejudice against the use of gravel before the introduction of machine mixing. The speed at which a mixer is run does not necessarily govern the time required to secure a proper mix, as it has been observed that when a mixer runs fast the concrete has a tendency to cling to its sides and does not travel properly through all machines.

A uniform gray color of the mix does not necessarily establish the fact that it has received sufficient mixing.

MIXED SAND AND GRAVEL.

Considerable difficulty was experienced in finding adequate storage for concrete aggregates, especially in the lower parts of Manhattan, where the streets are narrow and the traffic heavy. This, together with the increased cost of cartage and the difficulty of keeping the numerous mixers supplied with the proper quantity of each aggregate, led to mixing the coarse and fine aggregates in the proper proportions at the source of supply and shipping them ready mixed to the works.

At the present time there are two companies which ship the mixed aggregate. One company dredges from Long Island Sound, and separates the sand and gravel into various sizes, known commercially as $1\frac{1}{2}$-in., 1-in., $\frac{5}{8}$-in., $\frac{3}{8}$-in., coarse grits, bird sand and rubbing sand. These various sizes are then remixed on conveyer belts in such proportions so as to make specification sand and gravel, and in passing from one belt to another and through chutes are properly combined.

The other company mines its product from the bank, washes it thoroughly, and then separates it into specification sand and gravel. At this plant the sand and gravel are hauled in cars to a loading pier especially equipped for mixing and loading. There are two hoppers on this pier, one for gravel, and the other for sand, which receive the contents of the cars and discharge it on a conveyer belt. The bottom of each hopper is closed by a bucketed wheel, which is so geared that the gravel hopper empties at twice the rate of the sand hopper. The mixture is then carried on the conveyer belt to the chute which delivers it to the boat.

In order to prevent segregation in loading, the boats are moved back and forth under the chute, which is always kept in a vertical position, or as nearly so as possible, because when the mixture is fed to the boat from a chute at any other angle, the larger particles, possessing greater kinetic energy, separate from the finer particles. The combined aggregate is not allowed to pile up, but is spread uniformly over the boat in layers two feet deep. Field tests show that material on arrival at the mixer, after being unloaded from the scows and hauled through the streets, is very little segre-

gated, the proportions remaining practically the same as at the plant. In case of 1½-in. mixture, it has been found that if it has been mishandled or rehandled too many times, it may become segregated, while ¾-in. gravel, mixed with sand, may be handled quite freely without danger of serious segregation, hence we find that the greater the variation between the extreme sizes of the component parts the greater is the danger of segregation.

The proportions for concrete on the subway work, except in special cases, is 1 : 2 : 4. The mixture is, therefore, made in the proportions of one of sand to two of gravel; the ¼-in. square holed sieve being considered as establishing the dividing line between sand and gravel. We have found by tests in the laboratory and in the field that two parts of sand and four of gravel will yield approximately five parts when combined. Contractors, who receive a mixed aggregate, are, therefore, required to use one part of cement to five of the mixture.

DISCUSSION.

Mr. Humphrey. MR. RICHARD L. HUMPHREY.—I would like to ask whether there are any data available as to the relative strength of concrete placed by compressed air and that placed by ordinary methods.

Mr. Steinle. MR. J. G. STEINLE.—Our tests show but very little difference. The concrete placed by a pneumatic mixer may be a little more porous, that is, the fractures may show more air cells, but the concrete itself possesses about the same compressive strength. Our tests, which are more or less limited, extended over 28 and 90-day periods.

THE FLOW OF CONCRETE UNDER SUSTAINED LOADS.

By Earl B. Smith.*

Concrete as a material of engineering construction has been subject to a great deal of investigation along both physical and chemical lines, and as a result many valuable properties have been discovered and analyzed. The one property of concrete recently brought to light, which seems to be both necessary and sufficient to explain apparently abnormal stress and deformation phenomena, is that of flow under sustained loads. Flow is not used here in the sense of fluidity, but in the sense of molecular displacement or rearrangement under the influence of an external force. It is the gradual but persistent deformation which takes place in the material while in the condition of sustained stress. The analogy of flowing wax is weak and incomplete, but may serve to indicate what is meant by "flow of concrete." Concrete will not flow in the same manner as does the wax, but it will deform continually under stress up to a period of several months. This flow or continual deformation is not all permanent set but is partially elastic, the recovery being 25 to 50 per cent of the total deformation. The term, "time yield," has also been properly used to designate and describe this phenomenon, since a gradual yield does take place under stress conditions.

The discussion here is in connection with the flow or deformation in the direction of the primary or principal external force and does not refer to the lateral deformation. The physical explanation of this phenomenon and its possible control must be left to future investigations. It is the purpose here to mention only the fact that this phenomenon of flow in concrete exists, and to describe a few new experimental results.

Flow should not be confounded with shrinkage. Both factors exist in all concrete and they are easily distinguished. Shrinkage exists as a natural consequence, regardless of the size, position, or stress, and is influenced mostly by age and moisture. Flow, however, exists by virtue of sustained stress. It is a time-stress effect. Flow and shrinkage are distinguished by means of deformation readings on loaded and unloaded specimens as has been done in the tests described below. The curves shown herewith are for the net flow values only.

Compression Tests.

Concrete cylinders 3 in. in diameter and 24 in. long were loaded to 840 lb. per sq. in. by a dead-load applied through an I-beam as a lever. The concrete was 1 : 2 : 4 in proportion. Gravel and limestone were used in the cylinders under test, and companion cylinders were prepared for use without loading. In this way the contraction or shrinkage data for correcting the deformation of the loaded cylinders was obtained, thus giving the net flow

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deformation due to the load. The deformations were measured periodically by a Berry 20-in. strain-gage. These cylinders were all cured in the open air, and sprinkled once each day for the first week. The load was then applied after aging 28 days.

The results on the gravel and the limestone cylinders are shown in Fig. 1. It should be noticed, by reference to the "flow curves," that the flow is relatively great within the first week. The deformation increases here 100 per cent in four or five days, and requires nearly four times as long to increase another 100 per cent.

Particularly it should be noticed that the flow of gravel concrete is considerably greater than limestone concrete, yet the recovery is approximately the same. This relative flow of concrete as influenced by the character and kind of aggregate should be further investigated. The modulus of elasticity

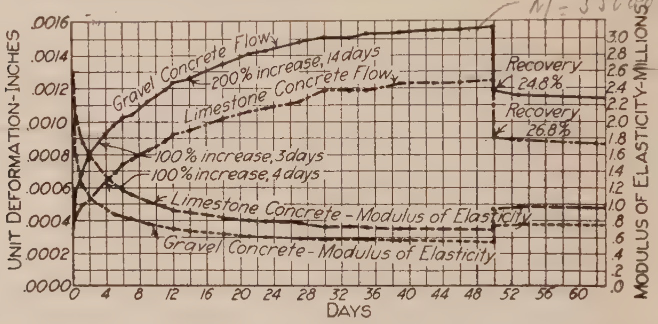

FIG. 1.—TESTS OF CYLINDERS.
Courtesy of the *Engineering Record.*

of these two specimens is shown by the two lower curves in Fig. 1. Here the modulus value as plotted is taken by definition to be the ratio of the unit stress to the unit deformation.

BEAM TESTS.

Two 5 x 8-in. reinforced-concrete beams were cast and at the age of four weeks were each placed on two supports of 10-ft. clear span and loaded at the center with a concentrated load of 830 lb. The effective depth was 7 in. and the reinforcement amounted to approximately 0.75 per cent, consisting of two plain $\frac{3}{8}$-in. square bars. The concrete was in each case 1 : 2 : 4. Gravel was used as the large aggregate in one beam and limestone in the other. Companion sections were cast of each mixture from which to obtain the contraction or shrinkage data.

The results of these tests are shown in Fig. 2. The upper group of four curves show the net flow in concrete and the increase of stress in the steel. The middle group of two curves shows the deflection of the beams; and the lower curves show the change in the position of the neutral axis.

The load was sustained on the gravel beam for 34 days, and on the limestone beam for 48 days. After these periods the load was completely removed. The concrete shows an elastic recovery after two days of 24 per cent for the gravel beam and 29 per cent for the limestone beam. The steel has apparently recovered to approximately its original condition before loading. The partial recovery of the position of the neutral axis should be noticed. This position was computed by means of the extreme compressive deformation values in the concrete and the tensile deformation values in the steel, assuming a straight-line variation.

1/4 ultimate

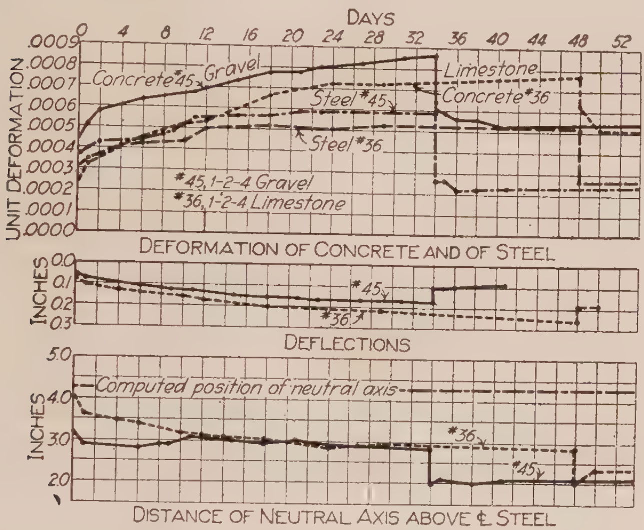

FIG. 2.—TESTS OF BEAMS.

Courtesy of the *Engineering Record.*

SUMMARY.

The law of the flow of concrete is asymptotic. The flow continues at a gradually decreasing rate and yet is an appreciable amount during three to four weeks. It then continues more slowly for an indefinite period, but this additional change is very small.

The natural total shrinkage of dry concrete is about 0.05 per cent in three months. The total net flow under load, exclusive of natural shrinkage, may be as great as 0.15 per cent, depending upon the time, material, and load. The total combined effects of shrinkage and flow in compression may amount to as much as 0.2 per cent. This would give, in a 20-ft. column not reinforced, if loaded to about 800 lb. per sq. in., nearly 0.5 in. of deformation; and may produce, in a reinforced beam of 20-ft. span fully loaded, a sag of nearly 0.3 in. Even the much smaller deformations, which are inevitable, may produce, if

not anticipated and provided for, serious results in the setting of apparatus and machinery, and in the alignment of shafting, and may easily cause other parts and members of the structure to be overloaded.

The effect of flow within the material itself is either to relieve the stress condition, if the construction and loading make this possible, or to gradually change the length or position of the member.

The maximum amount of flow or the flow for any particular period is almost directly proportional to the magnitude of the stress up to 1000 lb. per sq. in. (paper by the author in "Engineering Record," Mar. 4, 1916). It is, therefore, only necessary to decide upon the allowable flow deformations in designs before determining the allowable dead- and live-load stresses.

The measurement of stress conditions in concrete structures cannot be made directly by deformation readings, unless all the flow constants as to time, material, and loading are known. It is readily seen that deformation readings taken only a short time apart will indicate apparently different stress values.

The magnitude of the flow deformations vary quite largely with the kind of aggregate and the mixture. For instance, it is shown in these experiments that gravel concrete has about 20 per cent more deformation than limestone concrete.

The modulus of elasticity of concrete is different for each mixture and for each different aggregate. It changes and decreases in value with time and as the flow deformations increase. If the modulus of elasticity could be ascertained for any particular concrete with due respect to the time factor, stress values could then be determined by simple deformation readings.

In the case of a reinforced beam, the effect of flow in the concrete is to lower the position of the neutral axis, thus enlarging the compressive cross-sectional area and relieving the unit stress value. More stress is also thrown into the steel.

DISCUSSION.

Mr. D. A. Abrams.—It seems to me that we should be rather slow about accepting the comparative results Mr. Smith has presented in reference to gravel and limestone aggregates or whatever the aggregates may be. The concrete is made up of three different factors, cement, aggregates and water. As far as Mr. Smith indicated, the only factors common in these mixtures was the cement. If the aggregates are differently graded, or if the water is different, we may very properly expect different results, regardless of the character of the aggregate, so that if these factors are not taken into account, it is probable that the comparative results at least are misleading. **Mr. Abrams.**

Prof. W. K. Hatt.—I presume that this paper is one of several which evidences the plasticity of concrete. Concrete behaves like wood does; in other words, it deforms with continued loads, and the deformation seems to run out asymptotic after a certain period. We find in the case of wood, which exhibits many of the properties of concrete, a load continued indefinitely just above the elastic limit will finally break the wood. Beneath the elastic limit, it does not seem to produce much damage. I think that concrete has another property like that of wood in that the strength and physical properties are affected by the conditions in respect to moisture. We also know that wood is slightly affected by the temperature at which it is tested. These materials all seem to have different degrees of plasticity; they have different degrees of exhibition of colloidal properties. I wonder if anybody has ever experimented to determine to what extent the strength and deformations of concrete are affected by temperature and also by different degrees of moisture? **Prof. Hatt.**

Another thing that occurs to me—if these facts are as they are related, why don't we have more trouble in our buildings or in high concrete chimneys? Our building columns are loaded up to sometimes 1200 and 1400 lb. per sq. in. of reinforced concrete. Why don't these troubles come in on our concrete structures?

Mr. Smith.—In reply to Professor Hatt, I would say that if a column is heavily loaded and begins to sink in compression due to flow, it would undoubtedly throw different stresses in the crossbeams. Those beams may not crack or produce any dangerous condition, because they, in turn, will flow and relieve the stresses induced. In reply to Mr. Abrams as to the effect of the aggregate and water and the grading of the aggregate I think those things have an influence on this matter of flow, but how much, I don't know. They are simply factors in a test that have not been followed out yet and I think that this paper ought to be received, if at all, in the light of a progress report, simply as indicating a fact which does exist. **Mr. Smith.**

Prof. Hatt.—I understood the speaker to say that a stress of 800 lb. per sq. in. in concrete column would cause a deformation of about $\frac{1}{2}$ in. I would like to ask some of thse practical constructors if they ever get that? **Prof. Hatt.**

Prof. McMillan.

PROF. F. R. McMILLAN.—In this connection I will cite a case in which the glazed terra-cotta facing of a twelve-story building spalled off from the buckling action produced by the column shortening such as described by Mr. Smith. This was a progressive action and necessitated the replacement of portions of the facing several times. As there was positive evidence that

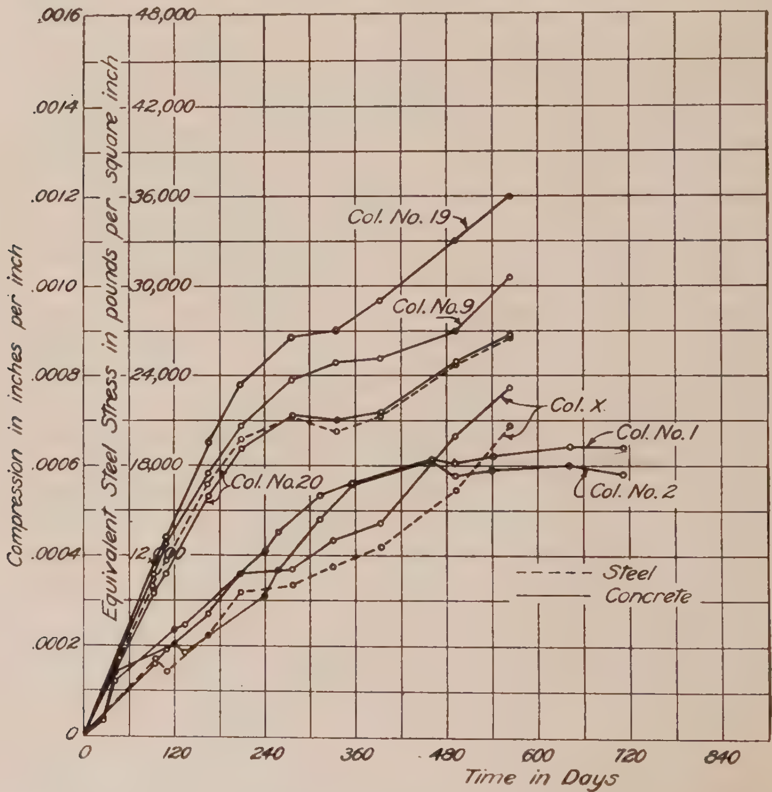

FIG. 1—TIME DEFORMATION TESTS OF REINFORCED CONCRETE COLUMN IN SERVICE.

(Observed by Prof. F. R. McMillan.)

the footings had not settled, the movement could be accounted for only in this manner. It was estimated that the total shortening from the shrinkage and the yield under load was about two inches for the full height of building. There is one circumstance in connection with the building of this structure that may account for the fact that the difficulty experienced is not noticed more frequently. It is the speed with which the erection progressed. The

terra cotta that required replacing was in place on the columns before they were two months old. Prof. McMillan.

To illustrate this same action the accompanying figure is presented. This shows the continuation of some measurements on reinforced-concrete columns which were presented at the last annual meeting of this Institute in my discussion of a previous paper by Mr. Smith. The results of measurements during the past year are represented by the portions between the 210th and 540th days on the curves for Columns 9, 19 and 20, and by the portions between the 360th and 720th days on the curves for Columns 1 and 2. The data for Column x were not shown previously.

These measurements were taken from buildings in service and are therefore not subject to the frequent criticism that they represent laboratory conditions only. Columns marked 9, 19, 20 and x are in one building in which the observations were begun about two months after casting and before all of the dead load from the floors above was in place. The movements therefore represent only a part of the dead-load deformation, a part of the shrinkage, and such time yield as has occurred in 600 days. The variation in the results for the several columns, ranging in value from 20,000 to 36,000 lb. per sq. in., is probably due to different conditions of loading and rates of shrinkage for the different positions in the building.

Columns 1 and 2 are in another building which was practically complete when observations were begun. In the results from these two columns, therefore, none of the dead-load deformation appears, nor does that part of the shrinkage which developed in the five months that elapsed from the time the columns were cast until measurements were started. Even under these conditions it will be seen that a deformation equivalent to steel stress of 18,000 lb. has taken place in two years.

The results from both these sets of columns are the more significant when it is considered that probably not more than one-tenth of the live-load for which these columns were designed has ever been in place, and then only for a few hours at a time, for these are university buildings in which the actual loads from the few score of students being accommodated is a small proportion of the full capacity. If these columns had been loaded to their full designed capacity, and measurements made to include all shrinkage and load effects, it is easy to believe that steel stresses of 40,000 lb. per sq. in. or more would have been shown.

PROF. W. K. HATT.—What is the stress on the concrete? Prof. Hatt.

PROF. MCMILLAN.—I have no means of knowing the stress on the concrete; those are the steel stresses. Only by knowing the exact load on the section and subtracting that carried by the steel would it be possible to get the concrete stress. Prof. McMillan.

PROF. HATT.—What type of columns are they? Prof. Hatt.

PROF. MCMILLAN.—They are spiraled columns designed as permitted by the Minneapolis Building Code; which, under ordinary conditions, allows 800 lb. on the concrete, 10,000 lb. on vertical steel, and 16,000 lb. on 2.4 times the area of the spirals. Prof. McMillan.

PROF. HATT.—What was the total shortening of the column? Prof. Hatt.

Prof. McMillan. PROF. MCMILLAN.—This must be calculated from the unit deformation. For a steel stress of 40,000 lb. per sq. in. the total shortening would be 0.016 in. per ft. In the column of the twelve-story building it was estimated that the total shortening between the sidewalk level and the roof was almost two inches.

Prof Hatt. PROF. HATT.—How was that determined?

Prof. McMillan. PROF. MCMILLAN.—By estimate based on measurements of small lengths; that is, not actual measurements with a tape line. Because the results are perhaps a little startling there may be an unwillingness to rely upon an estimate of this character, but I have no reason in the world to doubt the figures.

Prof. Hatt. PROF. HATT.—I have not either, but I would like to know how it was done.

Prof. McMillan. PROF. MCMILLAN.—It is a perfectly fair question and in view of the importance of the subject is entitled to a more specific answer. The estimate was made from the results of strain-gage observations on other structures, also from tape-line measurements of a seven-story building in which the movement was nearly a quarter of an inch per story. The results from the columns in the university buildings just described support the fairness of its estimate.

Prof. Hatt PROF. HATT.—If all these things are so, we ought to know about it, it ought to be evident in our constructions. If it is so, let us analyze it to see what they mean.

Prof. McMillan. PROF. MCMILLAN.—I think anyone who will take the pains to measure 20-ft. columns in a year or two from the time they are cast, will find a number of them shortened close to half an inch.

Mr. Ashton. MR. ERNEST ASHTON.—At what rate are the tops of the columns approaching the bottoms of the columns? If it is progressive it ought to be measured and we ought to know something about it.

Prof. McMillan. PROF. MCMILLAN.—About the same rate as was shown by Mr. Smith's curve, very rapid in the first few weeks, less rapid the next few months, and as time goes on decreasing gradually. This can also be seen from the curves of the columns just referred to.

SLAG AS A CONCRETE AGGREGATE.

By Sanford E. Thompson.*

The investigation and tests described in this paper are taken by permission from a report made by the author to Stone & Webster Engineering Corporation for the purpose of determining from the standpoint of its clients the availability of slag as an aggregate for plain and reinforced concrete.

The relatively high strength of slag concrete is well known, but few tests have been made to compare the relative value of different slags and to study comprehensively the various qualities.

The problems requiring research included:

(*a*) The use of slag for plain and for reinforced concrete.
(*b*) The relative value of slag versus gravel for building construction.
(*c*) The relative characteristics of slag made from different processes.
(*d*) The relative value of the same slag under different conditions of age, size and porosity.
(*e*) The strength of slag concrete and the permissible proportions and stresses to adopt.
(*f*) The durability of slag concrete.

Properly to treat these various questions, several series of tests were undertaken. The summary derived from these tests and from a study of previous investigations is presented below, followed by a description of the various tests.

Summary.

The results of the tests which are discussed in full below, may be summarized as follows:

1. The strength of concrete made with slag, such as is obtainable commercially in eastern and northern Ohio, was on the average about 50 per cent higher at the age of 28 days than gravel concrete made with first-class materials.
2. Using the same proportions by volume as for gravel concrete, about 15 per cent more cement on the average was required per cubic yard for slag concrete than for gravel concrete of the same proportions.
3. No authentic cases of deterioration of slag concrete made with portland cement or of rusting of steel imbedded in such concrete have been discovered.
4. Porous slag produced a concrete of substantially the same strength at 28 days as dense slag. At later ages, the dense slag is probably stronger.

* Consulting Engineer, Boston, Mass.

5. Slag made by different processes and under different conditions showed no marked difference in strength and other characteristics.
6. An extremely hard, dense, acid slag did not produce a concrete of greater strength than porous, basic slag on a 28-day test.
7. The weight of the basic slag concrete averaged about 6 per cent lighter than an average gravel concrete. On the other hand, very dense acid slag concrete was heavier than gravel concrete.
8. Granulated slag sand produced a mortar of inferior tensile strength on short time tests.
9. Crushed slag screenings produced a mortar appreciably higher in strength than standard sand mortar.

No tests of watertightness of slag concrete have been made in this series nor of protection of metal. Tests by other authorities and examinations of structures of slag concrete and tests made with concrete of other aggregates show that when properly laid the steel is protected from rust, even although the aggregate is porous.

Tests thus far made of permeability of slag concrete are insufficient to determine its availability for thin water-tight work, such as tanks.

The weathering qualities of slag concrete are indicated as satisfactory by examination of structures which have been built for a number of years, but further experimental investigations along these lines with different types of slag are needed before the conclusions as to its use can be considered final.

SPECIFICATIONS FOR USE OF SLAG FOR CONCRETE.

The tests have not been extended over a long enough period to warrant final recommendation for specifications. It appears from the results thus far obtained that for practical construction:

> Slag must be air-cooled, crushed, screened from dust, and free from foreign material. The weight of screened slag when shaken to refusal should be not less than 65 lb. per cu. ft.
>
> To insure a uniform dense mix, on account of the nature of the slag, exceptional care must be used in proportioning, mixing, and placing, especially where concrete is chuted.

The normal composition of commercial slags is given in the paragraph under "Chemical Composition of Slag." It is suggested for the present that slags not falling within the limits there given be rejected.

Commercial practice requires the banking of slags which are low in magnesia (say 1 per cent to 2 per cent) for at least six months. Slags of higher magnesian content (say 5 per cent to 6 per cent) appear to require a much shorter period of seasoning.

STRENGTH OF SLAG AND GRAVEL CONCRETE.

Table 1 gives the compressive strength of gravel and slag concrete made in the laboratory of the author. The proportions are by volume measurement, being made by dropping the measure four times from a height of about

3 in. The materials from Wellesley, Mass., were selected as a standard, but the ratio comparisons between gravel and slag are between concrete of Akron, Ohio, aggregates and slag aggregates, because Akron, Ohio, sand was used in the slag concrete, and the tests of this sand indicate a higher strength than Wellesley sand or than standard sand, probably because of limestone in its composition.

The various kinds of slag used in the tests are indicated in the table and represent ordinary ranges in commercial material. The acid slag, as indicated by the weight, is extremely hard and heavy and dense. The lime-

TABLE 1.—STRENGTH OF SLAG AND GRAVEL CONCRETE.

Proportions: 1 : 2 : 4 by volumes. Specimens, 6 x 12-in. cylinders. Age, 28 days.

Each value is an average of 3 specimens.

Aggregates.	Range in Size of Particles of Coarse Aggregate, in.	Weight, lb. per cu. ft.		Weight of Concrete, lb. per cu. ft.	Compressive Strength, lb. per sq. in.		Barrels of Cement per cu. yd.	
		Dropped Four Times.	Shaken to Refusal.		Average.	Ratio to Gravel.	Average.	Ratio to Gravel.
Wellesley, Mass., gravel and sand, commercial	$\frac{1}{4}$ to $2\frac{1}{2}$	102	107	151	2260	....	1.35	...
Akron, Ohio, gravel and sand, commercial	$\frac{1}{4}$ to 2	95	104	146	2780	1.00	1.38	1.00
Akron, Ohio, gravel and sand, commercial	$\frac{1}{8}$ to $1\frac{1}{2}$	96	102	143	2050		1.41	
Acid slag, banked, commercial	1 to $1\frac{1}{2}$	95	101	158	3410	1.41	1.61	1.15
Limestone basic slag, banked, commercial	$\frac{1}{4}$ to $1\frac{3}{4}$	74	77	143	3600	1.49	1.64	1.18
Limestone basic slag, green, selected	$\frac{1}{2}$ to 2	76	79	147	3940	1.63	1.64	1.18
Magnesian basic slag, commercial	$\frac{1}{4}$ to 1	68	69	148	3860	1.60	1.73	1.24
Magnesian basic slag, commercial	$\frac{1}{4}$ to $1\frac{1}{2}$	64	66	137	3700	1.53	1.57	1.13
Magnesian basic slag, picked dense	$\frac{1}{2}$ to $1\frac{1}{2}$	76	80	147	4070	1.69	1.62	1.16
Magnesian basic slag, picked porous	$\frac{1}{2}$ to $1\frac{1}{2}$	53	56	140	3850	1.60	1.41	1.01
Average slag concrete	......	..	..	142	3780	1.56	..	1.15

NOTE.—Basic slag is from blast furnaces in Ohio; acid slag, from blast furnaces in southern Ohio; magnesian slag, six weeks old; limestone banked slag, about six months old or more; limestone green slag, about three weeks old.

stone basic slag represents the type of slag containing a small percentage of magnesia (say 1 per cent to 2 per cent) which is run out into banks and allowed to cool for six months or longer. Green limestone slag is not sold commercially, and our sample was picked out for this particular test.

The magnesian basic slags contain usually from 4 per cent to 6 per cent of magnesia and represent the product from blast furnaces which use about 25 per cent of dolomitic limestone. This magnesian slag is used within two or three weeks after it is banked as there appears to be no chemical change in the slag on exposure.

Tests of the sand used with the slag, also tests of strength of mortars made with slag screenings and granulated slag are given later in this report as well as mechanical analyses of all aggregates.

Strength Versus Increase in Cement.—Inspection of Table 1 shows that the slag concrete averages 56 per cent stronger than the gravel concrete made with the same sand. Because of more voids than in gravel, a larger amount of cement is contained in a cubic yard of slag concrete than in gravel concrete of the same proportions by volume. This amounts to about 15 per cent, a greater difference than between broken stone concrete and gravel concrete which ordinarily is not over 5 per cent. Assuming that the strength of similar concrete varies with the percentage of cement, the net increase in the strength of the slag over the gravel concrete is about 36 per cent. In other words, even allowing for the additional amount of cement, the slag concrete is about one-third stronger than gravel concrete of the same proportions. Since tests made by the writer indicate that the strength of gravel concrete, such as is made in this series, is substantially the same as concrete of hard broken stone, the same ratios may be assumed to apply to stone concrete.

Cause of High Strength.—It is noticeable that the strength of the concrete is substantially the same whether or not the slag is porous or dense. This evidently is due to the fact that the solid part even of the porous slag is very strong and that the pores are filled by the mortar. Inspection of the crushed specimens where breaks occur through the pieces, shows a very strong bond between the mortar and the pieces of slag. While this probably is due in part to the rough nature of the surface of the slag, it is evident from inspection that a chemical action has taken place between the slag and the mortar. This is shown by a circle of a different color around the edges of the sections of slag. This possibly is iron sulphite produced by a combination of iron in the cement with calcium sulphite in the slag.

No difference is indicated in the table between tests of well seasoned slag and the green slag only a few weeks old. A part of the specimens made with the green limestone slag, however, gave exceptionally high strength and broke with a loud explosion. As already stated, this type of slag is not expected to be used commercially until it has been banked for several months.

Increase in Strength with Age.—While tests in these series have not yet been carried further than the 28-day period, experiments by other authorities indicate in general a normal growth in strength of slag concrete. An exception to this is where the slag is of extremely light weight, say, 50 lb. per cu. ft., in which case because of the low strength of this aggregate, the strength of the concrete on long-time tests does not appreciably exceed the strength at 28 days.

Weight of Slag Concrete.

The average weight of the slag concrete tested was 142 lb. per cu. ft. as compared with 144½ lb. per cu. ft. for the concrete made with the Akron gravel. The latter, however, is abnormally low for gravel concrete, possibly

because of the nature of the gravel, the weight of the Wellesley gravel concrete being more nearly representative at 150 lb. per cu. ft. Taking this as a basis, we find the average weight of slag concrete to be about 6 per cent lighter than gravel concrete, or 8 per cent if the slag is very porous, as is the case not merely with the picked porous specimen, but also with the commercial specimen of magnesian basic slag of $\frac{1}{4}$ in. to $1\frac{1}{2}$ in. size, which happened to be unusually porous.

STRESSES AND PROPORTIONS.

The Joint Committee on Concrete and Reïnforced Concrete base the allowable working strength on the strength of concrete cylinders 28 days old. If the Joint Committee recommendations are followed, therefore, it would be permissible to use with the slag concrete 50 per cent higher stresses than with gravel concrete, or conversely, to use lean enough proportions in similar ratio. While I am of the opinion that so large a stress is not yet warranted without still more extended tests, especially as a single series is not sufficient to establish definite ratios, the results indicate that where slag is accepted for use it is permissible to use at least 15 per cent higher stresses in the concrete than are normally adopted, or 15 per cent less cement with the same stresses, so as to put the slag concrete on the same basis commercially as gravel concrete. This would permit proportions $1 : 2\frac{1}{2} : 4\frac{1}{4}$ in place of $1 : 2 : 4$ commonly used in reinforced concrete. The slightly larger ratio of sand to coarse aggregate suggested is advantageous in filling the voids.

The modulus of elasticity is of importance in reinforced-concrete design and must be taken into account. No tests of elasticity were made in the present series. A few tests made several years ago at the Watertown Arsenal show about the same modulus for slag concrete as for stone concrete of the same compressive strength, and this relation may be assumed until tests determine this point.

PERMANENCE OF SLAG CONCRETE.

The chief objection to slag as a concrete aggregate has been based on the possibility of disintegration at some future time. To satisfy myself on this point, I have made chemical and physical tests and also visited several cities where slag concrete has been used for a number of years. As a result of a careful examination of various structures made with slag concrete, and interviews with users of slag concrete, I find no evidences of disintegration where portland cement was used. In certain cases with puzzolan, or slag cement, the concrete is not in first-class condition, but this is to be expected with any puzzolan cement concrete laid in air.

Chemical Tests.—Examination of limestone slag and comparison of its appearance where fresh, *i. e.*, two or three weeks old, and when seasoned by several months' exposure in the bank, show a change in color from a

distinct bluish or sometimes greenish tint to a much whiter color. Samples were analyzed qualitatively. The report of the chemist is as follows:

"These analyses were made on solutions obtained by quick treatment of the specimens with dilute acid so that these solutions would contain only the surface material.

"The main composition of all three is the same, that is, they show large percentages of silica, alumina, and lime, with a certain percentage of magnesia. That is, the main composition is undoubtedly calcium and magnesium silicates and aluminates. Each sample gives a slight test for iron but this is present in a very small percentage.

"The blue specimen shows a considerable amount of sulphide and a small amount of sulphate. The green specimen is similar to the blue except that the amount of sulphide is slightly less, the sulphate being similar in quantity. The white specimen, which represents the seasoned slag, shows a small amount of sulphide and a considerable amount of sulphate, also a decided evidence of carbonate.

"From these analyses it is evident that the change which takes place in color is due to the oxidation of the sulphides to sulphates. Probably most of the sulphide sulphur is present in combination with the calcium and it is also probable that there is a slight secondary reaction with the small amount of iron which causes the color.

"Pure calcium sulphide is light gray and would not account for the blue and green colors noticed. If the green color is observed after moistening, it is probable that this may be due in part to a hydration of the small quantity of ferrous sulphate, this compound being green in its hydrous condition and white in its anhydrous condition."

Calcium sulphate is a permanent substance. However, even if the slag is used before thorough seasoning, the particles are protected by the mortar so as to reduce danger of deterioration of the concrete. Magnesian slag is of a dark grayish color and more uniform in appearance. None of the sulphur compounds are of a nature that could act upon steel.

Accelerated Tests.—My experience has shown that the boiling test gives an indication of unsoundness not only in neat cement, but in mortars and concretes, and therefore provides some indication of possible future disintegration due to lime or magnesia.

I therefore made pats of the various mortars and boiled them. These showed no signs of disintegration or checking.

To extend this inquiry still further, I made tests of sand mortar specimens in the autoclave apparatus to compare mortar of slag screenings with mortar of good sand and found no appreciable difference in the result. The strengths of the specimens submitted to the autoclave test were high, and there was no expansion.

PROTECTION OF METAL.

No tests have been made in my laboratory of protection of metal. Tests in other laboratories, however, have shown satisfactory results, and the fact that tests of cinder concrete show that it will protect metal when

properly mixed and laid is evidence of the fact that slag concrete can also be depended upon when handled with proper workmanship. The appearance of broken sections of slag concrete is very dense and the ring of material already referred to which surrounds the porous pieces of slag tends to prevent penetration of water even through the individual particles.

WATERTIGHTNESS.

No definite tests are available on the watertightness of slag concrete. As stated above, slag concrete can be accepted as protecting metal from moisture and therefore can be considered as resisting the penetration of water. Further tests should be made before its adoption as material for tanks or other thin structures which must be watertight.

CHEMICAL COMPOSITION OF SLAG.

The chemical composition of slag produced by different furnaces varies. The material which we have designated as magnesian slag is apt to run about 4 per cent to 6 per cent magnesia. The limestone slag is apt to run from 1 per cent to 2 per cent magnesia. The compositions of slags are ordinarily included within the following limits:

Silica	32.0 to 36.0
Alumina and iron	11.0 to 15.0
Lime	40.0 to 48.0
Magnesia	1.0 to 7.0
Sulphur	1.0 to 1.7

MIXING AND LAYING SLAG CONCRETE.

Slag concrete does not flow to place so readily as gravel or even broken stone concrete, and cannot be used with so large an excess of water, nor does it flow down so flat a slope. These points, however, by no means prevent its satisfactory use in construction. The very fact that it requires extra care and will not flow readily tends to prevent the making of very weak concrete such as we so frequently find with first-class aggregates made with an excess of water.

TESTS OF MORTAR OF SLAG SCREENINGS.

Tensile tests were made of mortar with commercial slag screenings of magnesian composition and also by crushing samples into screening size, of seasoned and green limestone slag.

Tables 2 to 4 show the tensile strength of mortars of 1 : 3 proportion by weight and by volume, stored in air and water.

It will be noticed that the granulated slag weighs only 40 lb. per cu. ft., and the slag screenings weigh 84 lb. per cu. ft. This must be taken into account in making tests. It will be seen, however, that not only does the granulated slag give a very low strength when proportioned by weight, but

also when measured by volume the strength at 3 days is very low compared with normal sands, and indicates slow hardening. At 7 days the strength has gained, but is not nearly equal to the strength of mortar from slag screen-

TABLE 2.—TENSILE STRENGTH OF SLAG MORTAR IN COMPARISON WITH STANDARD SAND MORTAR—STORED IN WATER—PROPORTIONED BY VOLUME.

	Standard Sand.	Magnesian Slag Screenings.	Granulated Slag.
Per cent water	9	17.3	27
Proportion of mix	1 : 3	1 : 3	1 : 3
Briquettes tested 72 hours, lb. per sq. in.	216	246	132
Briquettes tested 7 days, lb. per sq. in.	227	319	231

TABLE 3.—TENSILE STRENGTH OF SLAG MORTARS IN COMPARISON WITH STANDARD SAND MORTAR—STORED IN WATER—PROPORTIONED BY WEIGHT.

	Standard Sand.	Magnesian Slag.	Limestone Slag Screenings.		Granulated Slag.
			Seasoned.	Green.	
Per cent water	9	14.8	12.5	12.5	19.5
Proportion of mix	1 : 3	1 : 3	1 : 3	1 : 3	1 : 3
Briquettes tested 72 hours, lb. per sq. in.	247	269	329	339	30
Briquettes tested 7 days, lb. per sq. in.	252	310	384	399	47
Briquettes tested 28 days, lb. per sq. in.	337	444	521	541	145

TABLE 4.—TENSILE STRENGTH OF SLAG MORTARS IN COMPARISON WITH STANDARD SAND MORTAR—STORED IN AIR—PROPORTIONED BY WEIGHT.

	Standard Sand.	Magnesian Slag Screenings.	Limestone Slag Screenings.		Granulated Slag.
			Seasoned.	Green.	
Per cent water	9	14.6	12.5	12.5	19.5
Proportion of mix	1:3	1:3	1:3	1:3	1:3
Briquettes tested 72 hours, lb. per sq. in.	243	267	275	310	25
Briquettes tested 7 days, lb. per sq. in.	272	381	395	352	30
Briquettes tested 28 days, lb. per sq. in.	260	467	446	400	84

ings. It is recommended that granulated slag be not used for mortar unless or until more extended tests determine its usefulness.

Mechanical analyses of the fine aggregates used in the concrete and mortar tests and of the gravels and slags are also given in Tables 5 to 7.

TABLE 5.—MECHANICAL ANALYSIS OF FINE AGGREGATES.

Size of Sieve.		Wellesley Sand.	Akron Sand.	Magnesian Slag Screenings.	Limestone Slag Screenings.		Granulated.
					Seasoned.	Green.	
		Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.
Inches.	No.						
0.50	$\frac{1}{2}$-in.	100.0	100.0	100.0	100.0	100.0	100.0
0.25	$\frac{1}{4}$-in.	98.0	99.0	96.0	79.0	87.0	97.0
0.16	5	96.0	97.0	91.0	70.0	79.0	95.0
0.0583	12	73.0	62.0	50.0	34.0	45.0	75.0
0.0335	20	52.0	43.0	36.0	25.0	32.0	51.0
0.0148	40	17.0	16.0	22.0	15.0	19.0	20.0
0.0110	40	10.0	9.0	17.0	13.0	16.0	16.0
0.0055	100	1.7	2.0	9.3	8.2	10.0	9.2
0.0030	200	0.42	0.5	4.0	4.8	5.7	6.9

TABLE 6.—MECHANICAL ANALYSIS OF COARSE AGGREGATE MAGNESIAN SLAG.

Size of Sieve.		Commercial $\frac{1}{4}$ to $\frac{3}{4}$ in.	Commercial $\frac{1}{4}$ to $1\frac{1}{2}$ in.	Picked Dense.	Picked Porous.
		Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.
Inches.	No.				
1.50	$1\frac{1}{2}$-in.	100.0	100.0	100.0	100.0
1.00	1-in.	97.5	78 3	30.8	28.0
0.75	$\frac{3}{4}$-in.	71.5	53.3	11.0	10.2
0.50	$\frac{1}{2}$-in.	32.3	17.7	3.1	3.0
0.25	$\frac{1}{4}$-in.	7.2	3.2	0.9	1.3

TABLE 7.—MECHANICAL ANALYSIS OF COARSE AGGREGATES.

Size of Sieve.		Wellesley Sand.	Akron Mixed Gravel.	Akron Fine Gravel.	Acid Slag.	Limestone Slag.	
						Seasoned.	Green.
		Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.	Total Per Cent Passing.
Inches.	No.						
2.50	$2\frac{1}{2}$-in.	100.0	.....	.....	100.0	.....	.....
2.00	2-in.	96.0	100.0	.....	95.2	100.0	100.0
1.50	$1\frac{1}{2}$-in.	83.0	92.0	100.0	47.8	91.5	96.2
1.00	1-in.	51.0	58.0	97.0	5.9	18.1	37.2
0.75	$\frac{3}{4}$-in.	34.0	34.0	90.0	1.5	5.6	11.7
0.50	$\frac{1}{2}$-in.	17.6	11.3	81.0	0.7	1.5	3.5
0.25	$\frac{1}{4}$-in.	5.5	1.68	29.0	0.4	0.8	0.9

FURTHER TESTS.

Further information is desirable on long-time tests of slag concrete exposed to air and to moisture to compare the relative value of porous and dense slag as an aggregate. Certain of these specimens should be ground so as to expose the aggregates to the weather. Permeability tests, tests of modulus of elasticity, and tests of expansion from changes in temperature and moisture also should be made.

DISCUSSION.

Mr. Ernest Ashton.—Will Mr. Thompson tell us what percentage of water was used? Mr. Ashton.

Mr. Sanford E. Thompson.—The consistency was substantially the same and it is the consistency which affects the strength. There was not enough difference in the consistency of the different concretes appreciably to affect the results. Mr. Thompson.

Mr. Richard L. Humphrey.—I would like to ask Mr. Thompson whether that slag all came from one place and how it was selected? Mr. Humphrey.

Mr. Thompson.—The slag came from three different places and from three different companies. It was selected by an engineer who was not in any way connected with the slag company. Mr. Thompson.

Mr. Humphrey.—Was any effort made to see what the variation in weight was for the different types of samples of slag? Mr. Humphrey.

Mr. Thompson.—The range of weights per cubic foot when measured by dropping the measure was from 53 lb. per cu. ft. up to 95 lb. per cu. ft.; when shaken to refusal the range was from 56 up to 101 lb. per cu. ft.; the 101 and 95 being acid slag. The range in the basic slag was from 56 up to 79 when shaken to refusal. Mr. Thompson.

Mr. Humphrey.—There are a good many objections to the use of slag, and while it may be true, as Mr. Thompson's tests show, that slag concrete does have great strength, yet there are difficulties in the way of the use of slag. I know that in Philadelphia, when they first began to use slag, they started with the weight per cubic foot and the amount of sulphur, as some indications of the measure of the quality; they got the sulphur content so high that they had difficulty in securing slag that would meet the requirement, while the variation in the weights of consecutive carloads of slag and of different portions of the same carload was quite great. It seems remarkable that a porous material of the same proportion in volume should give greater strength than a non-porous material. In a actual case where slag was used, in which I was an arbitrator, the contractor found that the number of batches from the mixer did not give the expected number of yards in place, and as a result of tests, it was found that there was considerable shrinkage in volume of the slag concrete in place. It was also found that it made a material difference in the mix and strength of the concrete as to the manner in which it was prepared. If the cement was mixed with the slag, it went into the pores and you got one result. If a mortar was made and then the slag added you got another result. Mr. Humphrey.

I realize today that there are a great many people making slag commercially and washing it and screening it and preparing it. I also have in mind a lot of old slag banks containing all sorts of material which is the

Mr. Humphrey. by-product of the manufacture of iron, and that material has a tremendous range in value, and I think that we need to go very slow in accepting, for the use of reinforced-concrete construction, material of this kind until we have more positive proof as to its value.

Mr. Brown. MR. H. W. BROWN.—I think that the great trouble with tests of this nature is that in the laboratory we get very good ramming in the cylinders, whereas out on the job there is likely to be considerable variation in the strength, and in a material of a porous nature, such as slag concrete, the amount of ramming is going to affect the strength very materially.

Mr. Thompson. MR. THOMPSON.—No investigation of density was made except in determining the weight and some rather approximate computations. The nature of the slag makes it extremely hard to go into that feature of density because of the pores.

Mr. Chapman. MR. CLOYD M. CHAPMAN.—I want to emphasize one point Mr. Thompson brought out, and that is the adequacy of mortar required to fill the voids in slag. I have in the laboratory verified most of these results which Mr. Thompson has mentioned. In a slag I tried to mix 1 : 2 : 4, the mortar would not fill the void. I went to 1 : $2\frac{1}{4}$: 4 and it would not fill the voids. I went to $1\frac{1}{2}$: $2\frac{3}{4}$: 4, which is about 1 : 2 : 3, before I got a concrete in which there were no voids, that is, in which the mortar was sufficient to fill the voids in the slag. That is a large excess of cement to be paid for if you want to use that particular slag.

COST ACCOUNTING FOR THE CONTRACTOR AND ITS RELATION TO HIS ORGANIZATION.

By Leslie H. Allen.*

A system of cost accounting to be of any value to a contractor must be used. Its data must be accessible to and made full use of by everyone in his organization who has anything to do with costs. It is valueless if it is kept a secret.

A proper system of cost accounting may be likened to an expensive tool or machine—of great value when working, but an unjustifiable expense if kept idle most of the time and not made full use of.

The attitude of many contractors towards cost accounting is like the attitude of the public towards the very nicely polished axes and saws that we see in glass cases on our railroad trains. I have never seen them used and I presume that their only use is to break open the windows in case a passenger desires fresh air. Such a remote contingency has never occurred in my traveling experience.

The contractor who thinks that his unit costs are a trade secret and the circulation of information regarding his costs will do him an irreparable injury is making a great mistake. If his costs are higher than his competitor's they will lose business if they use them. If they are lower they will lose money if they use them. It is efficiency and not cost data that makes for low costs. The cost data simply point the way to efficiency and show the failures to reach it. For this reason my firm has not hesitated to make public its costs at any time. We feel that if we make lower costs than our competitors it is because we have the men and the brains needed to do it. Others cannot equal our costs simply because they know them. On the other hand if our costs are higher than they should be no one gets any valuable information out of their publication that can give them an advantage over us.

The concealment of cost data is one of the reasons why it is so difficult to get accurate results. No employee can take an intelligent interest in work which is meaningless to him, or in laboring without seeing or knowing what the finished result will be. Very many contractors have the labor costs distributed by a timekeeper; this work is sent in to the office and worked up in his spare time by the bookkeeper; and the results are only seen by the boss. Neither man understands his work, or can show any enthusiasm for it, or cares very much whether it is right or wrong, provided that it will "get by." The opposite of this is the case on our work, and it is sometimes quite embarrassing to the writer to be called upon to adjudicate in cases where knotty points are being discussed with great earnestness by our timekeepers and foremen, such as whether repairing forms damaged by the strippers should be charged to stripping, erecting or making—or what to do with the

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saw filer. The fact that such interest is shown in minor points indicates a real interest in efficient work and low costs all down the line.

Some time has been spent in emphasizing the need of making cost data open and accessible, for if the contractor cannot be convinced as to the wisdom of this publicity the rest of this paper will be of no value. It is the purpose of this paper to show how the data furnished by a cost accounting system is made use of in the organization and supervision of the jobs and the carrying on of the general business of the contractor, and to demonstrate the vital necessity of up-to-date information on costs to the various departments of his organization.

If the contractor is not prepared to go thus far he is almost as well off without any cost system at all. He certainly is not getting half the return out of it that he should. This is true not only of a contracting business but of any manufacturing plant. Many cases have come to my notice of factories in which large sums are spent annually on an intricate cost accounting system, the figures of which are understood only by the cost accountants and shown only to the president or general manager, who in most cases understands less than a third of the data furnished.

The contractor's problem therefore in any system of cost accounting is not so to conceal the costs that no one but the boss can get them or understand them, but to make them accessible to and understood by the greatest possible number of people in his organization. The more people he can educate to understand cost analysis in his organization, the more intelligently and sensibly is his work going to be handled. Not only so, but it is very desirable that the architect or engineer should have some idea as to the costs of the work being done by the contractor under his direction. Many cases occur to my mind of incidents on our own construction work where the engineer's knowledge of what our work was costing has led him to allow modifications in the design which have saved a good deal of money for our clients, whereas if our cost accounts had been concealed our suggestions and arguments regarding a change in the design would have gone unheeded. For instance, on a recent job where the depth of column footings varied owing to bad ground, the engineer desired to build all footings the same thickness and to start all columns at the same height and to make up the different depths between top of footing and bottom of column by a pyramid of concrete, each one of which, of course, would need separate forms of varying shapes. Instead of arguing on a matter of opinion we were able after building two or three to demonstrate as a matter of fact that this method was much more expensive than to put additional concrete into the footings, and we were allowed to make the change.

If the contractor has been brought to the point where he is ready to admit that it is wise to make general use of his cost data in his business we now come to a discussion of the men who shall see this data, the reasons why, and the use they should make of the figures. Before doing this it is necessary to give a brief outline of the chief features of a contractor's cost system as follows:

First the estimate: (*a*) An estimate for each job based on the costs of earlier jobs of similar nature; (*b*) This estimate analyzed to show first the quantities, unit costs and total costs of each item of labor, and then the quantities and estimated prices of each item of material and each sub-contract.

Second: A daily labor cost report showing the labor costs of each day's work.

Third: A weekly labor cost report showing in parallel columns the estimated and actual quantities, unit costs and total costs of the work performed.

Fourth: A monthly statement of the cost of materials and sub-contracts purchased or ordered, showing in parallel columns estimated and actual costs.

Fifth: A final summary made at the close of the job showing the total cost of all labor and material.

The methods used by different contractors to secure this data may vary. In a paper presented by the writer before the Boston Society of Civil Engineers in March, 1914, the details of the cost accounting system used by the Aberthaw Construction Co. were fully presented. Although there has been a development and steady improvement in the system since, the main outlines remain unchanged. The object of the writer's paper at that time was to show that it was possible and profitable to set up a simple and economical system of cost accounting for a contractor and describe how this company had solved the problem. The present paper will take a further step and show how the cost system is used by the various departments of the business.

Taking the chief features of a cost accounting system in order, we come first to the estimate. What use should be made of this? In many offices it is locked up in the safe the day the contract is signed, and never sees the light of day again. The estimate is, or should be, the result of the estimator's study of cost data on preceding jobs, and is, therefore, the standard set for the cost of the job for which it was made. With it daily labor costs and material purchases should be compared. In making the estimate, the estimator has passed in mental review the whole sequence of job operations from start to finish and has described them in terms of quantity and cost. Read side by side with the plans and specifications it presents a complete view of the job from another angle and contains much valuable information that is most useful to, and should be made use of, by the contractor's force.

In order that it may be of the greatest possible use it should be in sufficient detail to show in separate items the various kinds and classes of work done by different gangs, and it is best to make an analysis of the estimate into two main divisions; 1. Labor. 2. Material, sub-contracts and all items not classed as labor. The total of the first division, therefore, gives the estimated total payroll and shows how it is expected that this will be spent upon carpenters, masons, laborers, etc. The second division shows in detail the quantities of material, sub-contract work, etc., item by item.

Who should see and use this valuable information and work to these standards?

First; the general superintendent. It gives him standard of performance by which he can measure up his job superintendents and foremen. It tells him more in a few minutes about the supply of materials, and the sub-contracts than a day's studying of the plans would, and in general is a big help to him in supervising the employment, supply of material, progress, etc.

Second: The job superintendent needs to see the estimate more than anyone. From it he can see what the firm expects him to accomplish in the way of costs, and his criticisms of the estimate should prove helpful to the estimate. With it he can confer with the purchasing agent of the company and with the local dealers from whom he has to buy supplies, and without a lot of computation on his part he can place the orders for much of his materials. From the sub-divisions of the analysis he can determine the sort of organization that should be placed upon the work, how many carpenter foremen, how many carpenters, the size of the labor gang, the number of mechanics, etc. Although in our company these things are checked up independently, yet we make the estimate the basis of any such reckoning. He can be sure that by reading the estimate it will call to his mind all the work that has to be done and no important item is likely to be forgotten until towards the end of the job, too late to be worked in economically with the rest of the work.

The estimate shows him the quantities of materials he has to receive and he can plan for the efficient storage in convenient places for them and confer with the purchasing agent and the owners of the building in order to arrange how the materials should be bought, when and where they should be delivered.

Third: The purchasing agent. This man does not use the labor division of the estimate very much, but the material section gives him a list of all the items that he has to take care of. From it he prepares a more detailed schedule and by taking dates from the progress schedule he can prepare for his own use a list showing all materials required and the dates required in order that he may start in buying intelligently and see that everything is delivered on time. He has a valuable check on the bids received, which is particularly useful when it happens to be impossible to get more than one or two bidders on an item.

Fourth: The scheduling men also should receive the material section of the estimate, they find from it what materials have to be scheduled, such as hard-pine framing, reinforcement bars, sash and other items on which an accurate buying schedule has to be prepared. They check their quantities by it. From the purchasing agent they receive notice of the date these schedules are required in order that he may buy them and have them delivered at the job in time.

Fifth: The foremen of brickmasons, carpenters, concrete gangs, etc., should be told what the estimated costs of their work are, and shown from week to week how their work compares with the estimate.

The second important feature of a cost accounting system is a daily report of the labor costs. These are best prepared on the job, and on a well-

run system should be in the job superintendents' hands by nine o'clock the next morning.

If each carpenter foreman is told what are the estimated costs on the operations he is to do, and he is notified every day as to whether he is coming inside or overrunning his costs, it is sure to have its effect. We frequently plot the principal items on a chart graphically which not only the carpenter foremen but the carpenters can see as they check in and out. We even draw similar charts for the concrete and steel work on occasion, and find that the lower laborer takes a cheerful and even enthusiastic interest in the vagaries of the wandering line that shows the cost of the work he is doing, and where the work is large enough to allow of gangs competing on similar work the graphic chart is even more valuable.

We believe that it is best to keep the cost accounts on the job instead of in the head office. Costs to be of any use should be fresh and not stale. If the superintendent knows at nine o'clock the very next day that concrete cost twenty cents a yard more to put in than on the preceding day he can talk it over with his foreman while the matter is fresh, and investigate and remedy the fault; but if the news does not come to him for a week or ten days after the work in question is done, it is too late to do anything. The daily reports come too often and in too much detail to be useful to the general superintendent. He would only be confused if he attempted to digest them.

It must not be supposed, however, that a system of daily cost reports will relieve the superintendent of any of his responsibilities. He is usually able to detect inefficient or expensive work by his own experience and observation without waiting for the costs to tell him. The daily costs do, however, point out an occasional high spot that he has missed. They are an unfailing barometer of the job. They strengthen his position with his men by backing up his judgment and are especially a very great help to younger superintendents.

The third essential of a cost system that has been mentioned is a weekly labor report. This should show in parallel columns the estimated and actual quantities, unit costs, total costs, and saving or overrun. The amount of detail will vary in different concerns, but in any case it should be in sufficient detail to give an intelligent account of the job operations. This statement should be furnished to the heads of the main office of the company as well as to the job superintendents, but accompanying it a simple summary should go to the heads of the company, the main facts being condensed into a very few items so that they can tell at a glance without a lot of study how each job stands. This statement gives a sort of review of the job's operations to date. It should go to the general superintendent and the estimator in the head office. The latter particularly needs to know the fluctuations in costs in order that he may check up his judgment with actual facts and learn the costs of any new sort of work.

The fourth essential feature of a cost system which I have mentioned is a monthly report showing in tabular form the estimated and actual costs and quantities of the materials, and amounts of sub-contracts and other expenses incurred. It is not necessary to have a weekly statement of this nature,

because differences between estimated and actual costs are not due to the good or bad supervision that the job is having, but are due to errors of judgment in estimating, unexpected rises or falls in the market, lack of information as to local costs, etc. It is, however, necessary that at least once a month track should be kept of the amounts gained or lost on the estimate. By adding together the savings or overruns on the labor and materials the contractor can tell every month just how far behind or ahead of estimate his job is, and can make an intelligent forecast as to how much can be saved or will be lost on the remainder of the job. Such information is rarely to be found in a contractor's office, but is of great value to him, and in itself is worth almost all the time and expense that should be put in on a cost accounting system. This statement should of course go to the job superintendent and to the general superintendent who are using the materials, and to the purchasing agent who spends the money, and to the estimator who finds it a most valuable check on his work, keeping him in touch better than anything else could with current prices. It is not enough for him to know six months after he has made an estimate that he has been wrong on some item. He should be kept constantly in touch in this way with the actual cost of the work that he is estimating.

The fifth essential feature is a final summary of the job costs made up in the same form as the original estimate, which should be filed for future use. A copy should be given to the job superintendent as a record of what he has accomplished, and to the general superintendent as a permanent reference for him, in order that he may have something to go upon in his criticism of future jobs. The estimator should have a copy as it is his chief authority for prices he places upon new estimates.

In the time available it is not possible for me to go into detailed methods by which these results can be accomplished. It must suffice to say that they can be accomplished without undue expense and trouble if the problem is handled in the right way, and a proper system of cost accounting will be found of inestimable value. It cannot be denied, of course, that cost accounting on contract work is a difficult problem, especially in the many cases where the job operations are many miles from the head office. It seems to me that if the factory owner finds it profitable to have a cost system when he is making a regular product on which his selling price is not fixed until he has made the goods and knows what they cost, it is of vastly greater importance to the contractor to have a cost system because he fixes his selling price first and finds out what the products cost afterwards. He is not even able to make it in his own factory under his own eye, but has to ship plant out on to the owner's property and make his goods, *i. e.*, the building on the owner's property, and then move his plant (which corresponds to his factory) away. Without some system of cost reports he cannot keep regular, systematic and proper checks on job efficiency and job expense, and unless he gets all his men co-operating in an intelligent way in an endeavor to find out what things cost and study the causes for the fluctuation, he is operating at a very great disadvantage.

DISCUSSION.

MR. FRANK R. WALKER.—After listening to Mr. Allen's paper and noting the completeness and thoroughness with which he has covered the entire subject of cost accounting on construction work, one is inclined to ask, "What is there to discuss?" Perhaps this has been brought home most forcibly to the writer because he is supposed to furnish the discussion. Mr. Walker.

However, it is true and deplorable that cost accounting is not generally used by contractors on construction work, but the fact is undeniable that those who have once tried it are still using it, and they find that the results obtained are worth far more to them in the performance of their business than the actual time and money expended in the compilation of same. This is especially true where the systems in use are practical and are not bound hand and foot with so called "red tape" that render them too complicated or expensive for use by the average contractor.

A practical cost-keeping system cannot help but be of great value, not only to the contractor, but to every member of his organization as well. It not only gives each man a keener insight into the actual construction of the work, but it teaches them to think in terms of actual performances instead of faint remembrances or vague possibilities. If a contractor makes an attempt to keep definite costs on the work he performs, at the completion of same he usually knows, in addition to what the work actually costs in dollars and cents, the special advantages gained or the unusual difficulties encountered and what conditions tended to lower or raise the unit costs, so that these methods may be either repeated or shunned on future jobs should the same conditions present themselves again.

By keeping in touch with the costs of the different classes of work, the contractor is able to suggest to architects, engineers, and owners the most practical and economical methods of designing and performing certain classes of work, and in this way win a contract that might otherwise be abandoned or delayed on account of the apparent prohibitive cost of same.

A knowledge of correct costs is as necessary to the success of the estimator as a working knowledge of financial matters is to the banker; because costs are the estimator's stock in trade, and he deals in them almost exclusively. Upon the estimator's judgement of the cost of performing all classes of construction work bids are submitted and contracts are taken, and upon his ability to a great extent depends the success or failure of the contractor by whom he is employed. If a mistake is made in pricing any portion of the estimate, the job must be started with a loss, and in the wake of present-day competition this is a difficult matter to overcome, no matter how efficient the organization on the job may be. In order to estimate the cost of construction work intelligently, the estimator should have an accurate knowledge of job working conditions; and as his occupation does not permit him to spend much

Mr. Walker. time, if any, on the job, actual costs properly compiled coming in from the job each week will do more toward keeping him well informed than any other method that might be used, and thus mistakes and "leaks" are stopped at once and are not repeated in future estimates.

A few years ago I knew a contractor who was constructing a large reinforced-concrete building, and during the progress of the construction the owners decided to add an additional story over a certain portion of the building. They asked the contractor to submit his proposal for performing the work. The contractor had not kept any detailed costs on the various branches of work in the original contract and so the cost of the additional work was estimated at a price which the contractor thought would allow him sufficient materials and labor to complete the new work and leave him a handsome profit. In due time he was awarded the contract for the additional work in accordance with his proposal and the work was completed. It so happened that the job organization was more progressive than the contractor and for their own information had kept a very accurate record of the labor costs on the job, both in labor hours and money. After the job had been completed the writer was given access to both the original estimate and the actual costs obtained from the job, and upon investigation it was found that the contractor had taken several thousand dollars worth of additional concrete form work at about 25 per cent less than the work had actually cost him, and this occurred despite the fact that 75 per cent of the original contract had been completed and the contractor had had sufficient opportunity to obtain accurate cost data on similar work already in place. The loss incurred on this one item alone would have more than paid the cost of maintaining an efficient cost-keeping organization on the job.

The man in the home office who purchases the materials, the superintendent, and the time-keeper on the job, should be furnished with a copy of the analyzed material estimate showing just how many cubic yards of concrete and the number of square feet of forms, etc., it requires to construct the foundations and each individual story of the building. By giving this information to the purchasing agent and the men on the job, they are able to buy all materials to better advantage, as the conscientious employee will always endeavor to purchase materials within the estimated amounts where possible. On the other hand, if this information is not furnished the job, the superintendent and time-keeper have no knowledge of the estim ted prices on the materials they are supposed to purchase, they are likely to obtain only one or two quotations on the materials and buy from the concern quoting the lower price. If they have the estimated allowance for each item to be purchased they will usually look further in their endeavor to purchase the materials within the amount named in the estimate.

By furnishing the job with the quantities of materials required to construct each story of the building, it gives the job an opportunity to check these amounts with the quantities of materials actually used, and thus, in the event of short weights or short measure of any materials, or mistakes made in the estimate, these items can be investigated at once and rectified before it is too late.

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By furnishing the superintendent on the job with a complete copy of the analyzed estimate that gives the quantity of work estimated to complete the foundations, basement, and each story of the building, together with the unit labor costs for performing same, it gives the superintendent a goal to work for, and he will exert himself to the utmost to at least equal the estimated labor costs and lower them if possible. If he has not been able to perform the works within the estimated costs, the contractor is aware of the situation immediately, and can investigate same and learn the reasons at first hand and remedy them, if possible, before there is any great loss sustained. Instances similar to this will often save the contractor many times the expense of his cost keeping system, and avert losses that would otherwise go undiscovered until the completion of the work, when it would be too late to rectify mistakes.

Quite naturally the various foremen on the job put forth their best efforts when they know an accurate record is being kept on the work over which they have charge, and as the foreman's advancement usually depends upon the results he obtains as a foreman, he generally tries to keep his work within the estimated allowance, and it results in the weeding out of any careless or inefficient workmen that might otherwise be retained on the job. If the foreman is informed that his work ran a little high the previous week and he is shown what he has accomplished and the cost of same, it acts as a stimulus to make him produce better results and overcome the handicap of the previous week.

When the superintendent and foremen are well versed in costs together with the quantities of work a man will perform per hour or per day, they are enabled to estimate with accuracy just how many days it should take to complete a certain piece of work and how many men will be required to complete it within a stipulated time. This is of great importance, especially where men are scarce and where it is necessary to import them from distant cities.

As a general thing the time-keeper employed on construction work is a young man who has had little experience in the business and is usually anxious to learn as much as possible in order that he may be advanced to foreman, superintendent, or estimator. After he learns the advantages to be derived from possessing a thorough knowledge of costs on the various branches of work performed on the job, he not only fits himself for a better position but is a much more valuable employee and will render far better service than the man who possesses no knowledge of the costs of the work being performed on the job. When the cost schedules are furnished from the main office, it is usually the time-keeper who watches them the closest to see that the estimated costs are not being exceeded.

For accurate cost keeping it is absolutely essential that the labor hours and costs should be distributed on the job, and it is highly important that the time should be distributed every day while it is fresh in the minds of the superintendent, foremen, and time-keeper. If it is put off from day to day, the notes will become "cold," new events will have crowded out the recollection of the older ones and it will not be possible to obtain a proper distribution

Mr. Walker. of the labor costs. If the labor hours are not distributed correctly, the resulting costs will be absolutely worthless. Inasmuch as the superintendent, foremen, and time-keeper are best qualified to distribute the labor time according to the cost keeping schedule furnished from the main office, it is necessary that perfect harmony prevail among them, or it will not be possible to obtain satisfactory results; neither will the cost data be satisfactory.

Perhaps the contractor performing the smaller construction operations varying in cost from $10,000 to $100,000, on which he has not been accustomed to employ a time-keeper will think that the cost of maintaining a cost keeping system is an unnecessary burden. But this is not the case, as on smaller operations the foreman or superintendent on the job is able to distribute the workmen's time just as satisfactorily as the contractor himself, providing he is furnished with a schedule showing just how the time should be subdivided. This will give the superintendent or foreman a working knowledge of the results to be obtained and a keener insight into the advantages to be derived from same. On the ordinary job this should not require more than 20 or 30 minutes a day for the man handling the work, and it is sure to prove of inestimable value to all parties engaged or interested in the compilation of same. On the smaller building operations the quantities of work in place may be computed once a week, and the general averages for the week and for the entire job should be handled so that they can be arrived at in a very few minutes. The quantities on certain lines of work may be arrived at from the actual quantity of materials delivered on the job.

As there are always new firms entering the construction business, they are all recruited from the ranks of officials, estimators, superintendents, and foremen of older established construction companies, and, therefore, the one thing that makes the contracting business of today the hazardous profession that is, and the one thing that has reduced competitive bidding to almost a dead loss basis instead of a reasonable profit basis, is the limited knowledge of costs possessed by a great many contractors and estimators. It is only by educating the employees and business associates of today the great importance of compiling correct cost data that the construction business will be raised to the plane to which it belongs.

Mr. Tubesing. Mr. W. F. Tubesing.—About two years ago five contractors in Milwaukee agreed to start an economic bureau so that at least the five of us would be figuring on the same quantities not only to start our estimates with, but also to help out on our costs. After two years we have now, instead of five, seventeen members belonging to this bureau which last year we conducted for the sum of $6,000. We now get a typewritten report from our quantity surveyor, which gives all the details of story, height, amount of concrete in each floor, each column and each beam. A complete steel list giving the number of pounds of the different kinds of bars is also tabulated, and below these are also given the stresses as required, so that in making up cost you can verify anything that you desire, even months after the job has been completed and there is a final statement on the work.

Mr. Turner. Mr. H. C. Turner.—I would like to compliment Mr. Allen upon the great value of his paper to all of us. It seems to me that he has conceived

the problem most thoroughly from the contractors' viewpoint, starting with the estimate and carrying through to the complete analysis of costs. As worked out by our company the system is similar but different in some respects. In our practice the estimator analyzes every estimate and a copy of the analysis goes to the statistician; that is, our cost dope also goes to our general superintendent. That is an analysis of the estimate made up to conform to the same classifications which will be used by the time-keeper in his reporting the time and labor on the work, and generally the working out of the unit cost is done by the statistician or his office, not by the job clerk. **Mr. Turner.**

On very large work we frequently put an extra man on the work to work out the job costs, but as a rule we depend on the weekly averages, and the statistician is the one who goes on the job and makes up the averages for the work during the week and goes to the books and classifies labor costs, having the quantities and the work performed, and classifies cost accordingly and works out unit costs. These unit costs are furnished to the general superintendent and to the superintendents, and the statistician keeps in his records, in parallel columns, the analysis of the estimates and the actual costs as they are being worked out, as the job proceeds. No copy is furnished to our estimator, but he is instructed to confer frequently with the statistician as to the cost of the work being performed, and he studies particularly the results on each job as completed for his guidance in making estimates on similar work in the future.

THE RELATION BETWEEN ENGINEERS AND CONTRACTORS.

By C. A. Crane.*

When a discussion is invited on a topic involving the relations between two parties affiliated by business, social or political ties, the suspicion is natural that those relations are not altogether harmonious and are capable of improvement.

On first thought the conclusion is pardonable that the relations between engineers and contractors are merely those that exist between buyer and seller—the relations which contemplate a mutual agreement for an exchange of values. And why isn't that all there is to it? Why should there be any difference in the relations between engineers and contractors, as between buyers and sellers? Sometimes there is not—and sometimes there is, or there would be no occasion for this discussion.

Now of course we know that there are times when buyers and salesmen disagree—that is inevitable where human agencies are involved—but in the main their issues are pretty clear-cut and susceptible of easy adjustment. The buyer knows what he wants when he goes into the market. The seller knows whether his goods can fill those wants, and having satisfied the buyer on that point, all that remains is to determine the compensation—the exchange of values.

That represents a contract in its simplest form, and no matter how complicated the article in demand or what efforts may be required to supply it, the element of contract—the exchange of values—is perfectly simple, *when the buyer knows exactly what he wants,* and conveys that knowledge to the seller.

That's all there is to a contract, which has been defined by Blackstone as "an agreement upon sufficient consideration to do or not to do a particular thing," and one of the essential elements without which an agreement is not a binding contract is that there shall be a *mutual understanding* and a *meeting of the minds* of the contracting parties.

That's the stumbling block that causes most of the trouble in contracts—that is frequently responsible for the rift in the pleasant relations between engineers and contractors. When the buyer doesn't know exactly what he wants and leaves it to the seller to find out, there certainly has not been a meeting of the minds essential to a perfect agreement.

Now there is a vast difference between contracts to buy and sell a staple commodity and contracts for the construction of a building, a subway, reservoir or other public utility. Of course there is. In all large undertakings it is practically impossible to foretell precisely what may be wanted—what will be best to meet conditions that can be revealed only as construction progresses. We all know that. No engineer would be foolhardy

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enough to gainsay it. No contractor would be so unreasonable as to expect it. What contractors do object to, however, is the attempt of the engineer to shift the burden of this uncertainty entirely upon the contractor's shoulder in contract clauses reading like this:

> The foregoing engineer's estimate of quantity of supplies and the nature and extent of the work required though stated with as much accuracy as possible in advance, are approximately only; bidders are required to submit their bids upon the following express conditions, which apply to and become a part of every bid or estimate received. Bidders must satisfy themselves by personal examination of the location of the proposed work, and by such other means as they may prefer, as to the accuracy of the foregoing statement, and they shall not, at any time after the submission of their bid, dispute or complain of such statement or estimate of the engineer, nor assert that there was any misunderstanding in regard to the nature or amount of work to be done or the material to be furnished.

Or as found in another contract in shorter form:

> Neither the Department of Bridges nor the city is to be held responsible for the estimates of the quantities of material to be furnished or work to be done. The contractor has judged for himself as to such estimates as well as to the conditions to be met which will affect both the cost and time required for the execution of the work, and assumes all responsibilities therefor.

The guarded language in these clauses indicates that the buyer, in this case represented by the engineer, either was not sure as to just what he wanted or was unwilling to commit himself. Yet contractors are asked to submit bids on these estimates and frequently the time given them to inspect the site of the proposed work and verify the conditions does not exceed ten days. Engineers who have spent months in taking borings, making sub-surface examinations and preparing their plans and estimates know that it is impracticable, if not impossible, for a contractor to verify their work before bidding, even when, in some of the larger undertakings, a period of a month or six weeks is given for the examination. Is it not an evidence either of lack of ability or lack of confidence on the engineer's part to resort to this disclaimer? Self-preservation is the first law of nature, but fortunately the courts have not always construed that tendency as one of the components of contract law.

This question of estimated quantities has been litigated many times, and in several instances the courts have delivered very forcible opinions concerning it. There was the case of Robbins *vs.* City of New London.

The Water Commission of that town had advertised a contract to build a dam. The information to bidders contained the usual clause that the estimated quantities were approximate and that the bidders must satisfy themselves of the conditions. Shortly after starting the work, the contractor discovered that the original quantities were grossly under-estimated—not

always an unpleasant discovery—but in this case, the difference in quantities was such as to require entirely different plant than was originally contemplated. Because of the delay incident to installing new plant, and the discussion which arose with a view to securing different unit prices for the work, the engineer, pursuant to the authority in the contract, ordered the contractor to discontinue work and suit was brought to recover on the bond given for the faithful performance of the contract. The court dismissed the suit against the contractor and in the course of its opinion defined the real character in law of these statements of estimated quantities.

The opinion stated that the representations in the advertisement and the notice as to quantities carried with it "the assertion of being made upon some basis of superior knowledge. Their purpose was to supply information to persons who were expected to act upon it in a business dealing with those who made it, and who were entitled to accept and act upon it as expressing what it purported to express, to wit, information having a basis in such superior knowledge." Elsewhere in the opinion the matter is explained in other words: "It is apparent that the facts involved in the statements and representations in question were such as not to be equally available to both parties, were not at hand or within the observation of the contractors, and involved investigation of conditions, study and computations for which expert technical knowledge was required. They were made by a party in a position to have, and who assumed to have, not only a superior knowledge, but also a knowledge which had a foundation in expert examination and study, and they were made for the purpose of being acted upon, and promptly acted upon."

A more recent case is that of the firm of contractors, Leary & Morrison *vs.* City of Watervliet, which was decided in one of the New York supreme courts last October. This contract was for a sewer and contained the usual clause about the quantities. Forming a part of the contract were drawings showing profiles of the sewer on which was a line marked "approximate rock line." As the work proceeded, rock was discovered where none was shown on the plan, of a maximum depth of 14 to 16 ft., and this in a part of the town where the work was the most difficult owing to the proximity of buildings. The contractors sued for the extra cost of this work, and in awarding them judgment, the court expressed itself in this language:

> A contractor should be held to a strict compliance with the terms of the contract in the doing of all work according to the terms of the contract as illumined and explained by the maps, drawings, plans and specifications. The contractor should take every reasonable precaution to advise himself as to what he is undertaking and as to what he is reasonably expected to do.
>
> The city on its part should take every reasonable precaution and opportunity of informing itself through its engineers and experts as to what are the true conditions under which the contractor will be called upon to perform, also on its part to do in the way of preparation all that it reasonably undertakes to do, to the end that the contractor will not be required to do what he could not reasonably be expected to do.

In other words, it is quite as important that a municipality should be held to as strict an accountability on its part as is the contractor, and that the city should not receive the benefit of work done and materials furnished by a contractor, through a technical construction of the terms of a contract, all to the advantage of one party who had the greatest opportunity to know what was actually required and expected to be done.

The city fairly should be bound, to a reasonable extent at least, by the drawings which it presents to the contractors for their information. The city has every possible advantage in knowledge as to the conditions which exist, and, while it calls upon the contractors to find out for themselves, it can hardly be said that that means that the contractors should actually excavate before they begin their contract in order to find out what they may thereafter be called upon to do.

If the city says that rock disappears from the surface and is not included within the line of excavation at about this point or that point, it should be held that the city meant about what it said.

These two are not by any means all of the cases ruling on the representations of the advertised quantities and information. The highest court in the land, the Supreme Court of the United States, has had the question before it on several occasions, two of which are worthy of mention.

In Hollerbach & May *vs.* United States, decided in April, 1914, the contractor sued to recover increased cost of removing a dam because the material encountered was entirely different from that as represented in the specifications. Mr. Justice Day in reversing the Court of Claims remanded it to that court with directions to enter judgment for the contractors for the damages incurred because of the difference in character of material, and said in part:

> A Government contract should be interpreted as are contracts between individuals, with a view to ascertaining the intention of the parties and to give it effect accordingly, if that can be done consistently with the terms of the instrument. In paragraph 33 the specifications spoke with certainty as to a part of the conditions to be encountered by the claimants. The specifications assured them of the character of the material, a matter concerning which the Government might be presumed to speak with knowledge and authority. We think this positive statement of the specifications must be taken as true and binding upon the Government, and that upon it rather than upon the claimants must fall the loss resulting from such mistaken representations.

And again, in Christie *vs.* United States, decided April 12, 1915, a case which hinged on erroneous borings, a paragraph in the specifications provided:

> The material to be excavated, as far as known, is shown by borings, drawings of which may be seen at this office, but bidders must inform and satisfy themselves as to the nature of the material.

These borings showed gravel, sand and clay of various descriptions and showed no other materials. As a matter of fact, the material proved to consist largely of stumps below the surface of the earth, buried logs, cement, sand and gravel, all of which was much more difficult and expensive to excavate than the ordinary sand and gravel, as described by the borings.

Justice McKenna in delivering the opinion of the court said:

> There was a deceptive representation of the material and it misled. There were representations made which were relied upon by claimants, and properly relied upon by them, as they were positive. Besides it was admitted at the argument that time did not permit borings to be made by claimants.

The language in these opinions could hardly be stronger in denunciation of this attempt to shift responsibility, but mark this—the relief eventually obtained was not from the engineers who after the work was done, were certainly in a position to realize the full extent of the damage their mistakes had caused. No, it was at the hands of the court, and as some of you perhaps may know from your own unfortunate experiences, it costs money to appeal to the courts. In other words, the contractor was put to considerable expense to obtain what he was justly entitled to and should have received without litigation had the engineer done his full duty, either in the first place in properly preparing his estimates and plans, or in the second place, in acknowledging his error and certifying the amount for payment. That failure to make reparation is an unpleasant phase in the relationship between engineers and contractors.

I spoke of an increase in quantities not always being unpleasantly welcomed by contractors, *when they get pay for them*, but there have been instances where such increases, under the unit system of payment, have proved very detrimental to the contractor's pocketbook. When time is of the essence of a contract it is reasonable to assume that the estimated quantities will be sufficiently accurate to indicate that the engineers have devoted enough preliminary study to their design and estimate to know that the work can be done within the time prescribed. As a matter of fact, nearly all contracts provide that time is of their essence, but they usually provide that if subsequently some of the items are very largely increased, the time for completion will be extended commensurate with the increase. That is such a plain business proposition that one would hardly suppose any other course would be taken. It is the delightful uncertainties in contracting that make it so fascinating.

Winston & Company had a contract to construct a reservoir in the City of Pittsfield, Mass. They were to do the work within a certain contract time, and liquidated damages for non-completion within that time were $75 per day for the overtime. A detailed estimate of the quantities involved in the work was issued to the intending bidders. One of the principal items in the work was earth excavation, known as stripping. The engineer's estimate called for 37,000 cu. yd. and the actual quantity excavated was 96,000 cu. yd. Other items of earth excavation were estimated at 12,200 cu. yd.

and the actual amount was 24,700 cu. yd. Because of these increases the contract was not completed within the schedule time. In fact, because of the increase, the work was prolonged through the winter months when it was impossible to do little, if anything, but in spite of this very glaring error in the engineer's estimate the contractor was held to the original time and the penalty of $75 per day for 8 months, over $15,000, was deducted from his final payment. You can imagine how that contractor would characterize the relation between that particular engineer and himself. The suit to recover these overtime charges is still pending in the Massachusetts courts.

These cases all hinge on under-estimated quantities. Severe losses may be incurred from over-estimated quantities, and a moment's thought will convince you of this. In figuring a piece of work, the contractor must take into consideration a number of items, and very important items which do not appear in the engineer's plans or in his estimate of quantities. I refer to the overhead, and in one of the engineering journals sometime ago there was a list of 16 items comprised in this overhead, and that was a very conservative list. It included: 1, Expense of making up the estimate and bid; 2, cost of surety bond; 3, cost of liability insurance, workmen's and public; 4, cost of plant and equipment; 5, moving plant; 6, maintenance of equipment, including cost of repairs and supplies; 7, tools lost, broken and stolen; 8, wasted material; 9, demurrage; 10, losses due to weather conditions and damages by the elements; 11, payroll and expense of outfit in rainy weather; 12, water charges; 13, damages to private property; 14, cost of inspection of materials; 15, lost and damaged cement bags; 16, being a good fellow. To this list might be added, depreciation of plant, fees for various local permits, interest on borrowed capital, and one very important item which a contractor nowadays would be very foolhardy to omit, annual retainer to counsel.

Now this overhead, plus the profit, must be added to the cost of labor and material, and when unit bids are asked on itemized work, it would seem almost an office boy's job to apportion this overhead to the various items enumerated in the engineer's estimate. That would work out beautifully if the engineer's estimate were correct, but if the amounts of these items fall below the estimated quantities, a certain proportion of the overhead expense is lost. This is not at all an unusual experience, and to protect themselves the contractors have evolved a system known as "unbalanced bidding" in which they endeavor to place the greatest proportion of this overhead and profit on those items which their investigations lead them to believe are tolerably accurate.

There may be some excuse for the engineer to under-estimate his quantities, but the only reason for over-estimating to the extent which has unfortunately become very prevalent of late, even among engineers of the highest repute and standing, is the eagerness to keep the final cost within their estimates. This avoids the embarrassment of asking for extra appropriations and permits laudatory reports upon completion of the work calling attention to the fact that it cost less than the contractor's bid.

There is a simple solution for this matter of overhead, and that is to

include in the bid a separate item to cover the overhead charges, say a lump sum for the entire job to be paid in proportion as the work progresses. If this scheme were adopted it would remove the incentive for unbalanced bids, and at the same time assure to the contractor the actual expense that he was put to even though bidding quantities were considerably overestimated.

Another very prolific source of contention on contracts is "extra work." These are words that are presumed to roll pleasantly in the mouth of the contractor—and are viewed with suspicion by the public. Like charity, they have covered a multitude of sins, and reputable contractors are paying, and will probably continue for a long time to pay for the sins of their less scrupulous brothers. But don't lose sight of the fact that there must have been two parties to every fraudulent bill of extras—and if a contractor was one, wasn't an engineer the other?

Perhaps it's due to this dread of suspicion that engineers are so reluctant to pay for extra work—and perhaps, with a great many of them, there is a natural hesitancy in admitting a mistake and an omission, with the attempt to shield themselves behind the statement that the quantities were only approximate and the work was implied in the plans.

There are, however, any number of cases in which the engineer has ordered the contractor to do extra work, and when the bills were rendered they were not paid because the engineer had no authority under the contract to issue such orders. Here was a dual mistake—the engineer assumed he was empowered to order the work, and the contractor assumed the engineer knew his powers and accepted the order in good faith—with this difference, the engineer's mistake cost *him* nothing, while the contractor's mistake in some instances has cost him thousands of dollars. Moral for the contractor—don't ever do any extra work unless it is provided for in the contract, and then only on the order of the party authorized to issue it.

On big work, however, it is recognized that extra work may be legitimate and necessary, and we find in many contracts provisions for paying for it. This is the way a Philadelphia contract handled it:

> EXTRA WORK.—Whenever, in the opinion of the Director of the Department of Public Works, it shall become necessary to use materials or perform labor which is neither contemplated in the plans of the work, nor implied in the specifications referring to said plans, the contractor hereby agrees to furnish such materials and perform such labor as extra work, and agrees to accept in full payment therefor a price which shall be fixed by the Director of the Department of Public Works.

That may be better than nothing, but it's a long way from perfect. It's not fair to the contractor as it binds him to accept any arbitrary amount, and it's not fair to the public official who might sometimes be suspected of treating one contractor better than another.

On the New York subway contracts, extra work—that is, work not susceptible of classification under the schedule of unit prices in the bid—

is paid for at the net cost of the labor and material, and, quoting from the contract "in addition thereto 10 per cent of such net cost for the use of tools and plant, superintendence and all other expense incidental to the performance of such work and the furnishing of such material."

Extra work on that basis means an actual loss to the contractor. Note that 10 per cent is allowed, *not for profit*, but for tools, plant, superintendence and *all other incidental expenses*. It's a very unusual job where general expense, including those overhead items just referred to, totals as small as 10 per cent of the contract. The insurance alone, under the Compensation Law, on subway work is 14 per cent of the payroll, and insurance is not allowed, except in a few of the very recent contracts, in computing the net cost of labor.

So that method of paying for extra work is on the basis of half-a-loaf being better than no bread—and the more extra work the contractor does, the more money he loses. It's a long way from a cost plus 10 per cent proposition—and that is really the only fair way to pay for extra work, including in the cost the overhead expenses which apply as much to labor and plant on extra work as to any other part of the job.

Probably big construction contracts always will be more or less of a gamble, but as contracting is becoming more and more an exact science due to the presence of so many of our best engineers in that field, it would seem that many of the gambling elements could and should be eliminated. Take for instance the construction by the Government of the compensating dams along waterways that are especially susceptible to floods. These Government contracts generally call for a temporary cofferdam to be used in the construction of the work. The contract provides for payment of one cofferdam. If it be washed away, it must be replaced at the contractor's expense, and all other damages sustained by the floods must be borne by the contractor. Because of the inherent gamble in this class of work, there are not many contractors in the country anxious to bid, and such as do, make very careful studies of average weather conditions in the watersheds which would be liable to affect the locality where the work was to be done; but the weather sometimes shows utter disregard for averages, and the result is that on some of these Government dams contractors have lost big sums of money. They take a gambler's chance on average conditions. If the conditions subsequently are more favorable than the average, the contractor makes a handsome profit, and if they are worse, as in the case of some recent work on the Ohio River, where eleven heavy floods, some of them reaching a stage of more than 40 ft. above normal, occurred within a period when not more than one or two floods were generally to be expected, the contractor loses heavily.

It is unfair to the Government to have to pay more for its work than its reasonable value, and it's certainly unfair to the contractor to have to lose hundreds of thousands of dollars through agencies beyond his control. These contracts should provide a method of payment for flood damage. If there were no floods the Government would not be paying the big contingent overhead, and if there were, the contractor would be reimbursed

merely for his loss. Public works are being constructed for the benefit of the community at large. If any damage occurs because of these unforeseen acts of nature, the community which is to benefit by the work should assume payment of the damage.

Contractors for big work assume a big responsibility—responsibility for the lives of their workmen and for the lives and property of the public. Add to this the financial responsibility incident to the work itself, and it would seem that he assumed enough without taking on responsibility for acts of the Almighty. As a matter of fact, on public work, contractors are agents of the public, just as are the engineers over them. They are both on the same payroll, and each is entitled to his pay if he earns it.

Instead of Government engineers regarding contractors who do Government work as enemies to the flag they might with profit to the country give some study to a more equitable form of contract.

In striking contrast to these contracts under the War Department, is the attitude of the engineers of the Navy Department who are now engaged in just that study, and as an evidence of their announced desire to draft a mutually satisfactory document, they have invited criticisms of the present forms from contractors and builders, and their suggestions for improvement. This tokens a spirit that, if followed by the engineering profession generally, would go a long way toward bettering the relations between engineers and contractors.

Now I hope no one will construe my remarks as an indiscriminate arraignment of engineers. That would be not only highly impolitic, but quite the reverse of my real feeling. As individuals, engineers are the finest lot of men in the world—but they are clannish—and as a clan they are sometimes apt to get behind their contract clauses and sit tight. And no one criticises them louder than other engineers who have forsaken public employ and embarked in the contracting business themselves, when they get a taste of their own medicine.

If it's a service to mankind to make two blades of grass grow where one grew before, does not engineering genius render equally essential service in making one pound of material do the work where two were required before? Look at the advances in engineering design and construction within the last few years since the adoption of reinforced concrete. The saving in building and heavy construction costs over former methods has been enormous. The engineers have succeeded in making one dollar today do the work which yesterday required two, but present day costs of work are no less than they were before. The costs of labor and material advance with the times and the utmost engineering skill is called upon to devise methods and machinery to offset these advances. Public improvements would be prohibitive if this skill were lacking.

Engineers in charge of construction work—the buyers for the public—can effect an enormous saving in their lines in other ways than perfecting economical design. And one of these ways is in perfecting an up-to-date form of contract eliminating the present one-sided and unfair clauses, and adding what we might call "inducement" clauses—inducements to the many

reputable contractors who either won't bid at all on public work under present conditions, or bid extravagantly high when they do. The American Institute of Architects did that very thing a few years ago and their Standard Contract form is in universal use on private construction. And of whatever benefit accrues to the owner who pays for the benefit, a considerable portion is due to the efforts of the builders through whose persistent efforts the document was brought to its present state of perfection.

There's the point! Don't forget that the owner pays. Right here let me quote a pertinent paragraph from a pamphlet issued by the National Association of Builders' Exchanges in their campaign to secure reforms in the architects' contracts:

> It may be inferred that it is only for the benefit of the contractor that a reform is demanded. The fact should not be lost sight of that the contractor is not the ultimate consumer in the case and that he generally will protect himself from such conditions by sufficient allowance in his price; it is the owner who must eventually bear the burden.

Unfair clauses, ambiguity in specifications or quantities, lack of inducement, all combine to pile up the cost of work. The personal equation of the engineer, too, is taken into consideration by the contractor in preparing his bid. A prominent railroad contractor—a man who has built hundreds of miles of the most difficult railroad sections in the country was speaking of this point just a few days ago. He mentioned two of the big railroad systems which were not many miles apart and said that when the A. B. & C. road wanted bids he went after the work hard and bid to get it. But when the X. Y. & Z. road let a contract he didn't care whether he got it or not, but if he did his bid was 20 to 40 per cent higher than on the A. B. & C. The engineer of the X. Y. & Z. had a reputation among contractors for being the meanest, most arbitrary and hard-to-suit customer in the country. And, said he, it cost that road millions of dollars more to get its work done than the neighboring road paid, and for years to come it will be paying dearly for the reputation of its chief engineer.

Of course the owner pays. Talk to engineers about these things and they all admit the truth of it, but claim that the interest of the owner or the public must be protected against the dishonest or irresponsible contractor. Why, you can't hurt that fellow! He'll wriggle out of it somehow and leave the job to be finished at additional cost—and perhaps recover heavy damages in the end for having been thrown out. The reputable contractor cannot be bull-dozed with club clauses—he may take punishment, but if he finishes the work his chances are better than ever for a recovery in the courts. And the owner pays that bill.

Now, by inducements, don't misunderstand me as meaning a letting down of the bars and a free invitation for every Tom, Dick and Harry to come in and run the job to suit himself. But rather, inducements that will be an incentive to good and rapid work—inducements that will not only benefit the contractor, but accrue as well to the benefit of the public. Let me cite an example:

We have spoken of liquidated damages to be charged against the contractor for not completing his work on time. It isn't called a penalty any more, because the courts have held that you can't collect a penalty unless you also offer a bonus. So the lawyers get together and say we will stipulate that time is of the essence of the contract, and make the contractor agree in advance on a sum to be fixed as "liquidated damages"—that's not a penalty on the contractor, it's merely reimbursing the owner for the actual loss due to non-completion. Suppose the contractor can finish ahead of time, is it fair for the owner to delay him, and make him maintain his plant and force when he could be using them to advantage elsewhere, because the completed work is of no especial benefit to the owner ahead of time? Isn't he entitled to liquidated damages for the owner's delay?

Here's an instance of that to show what I mean. On one of the large aqueduct tunnels in New York the contract time was 48 months. By good management and improved methods the contractor had completed 95 per cent of the work in 26 months. The remaining 5 per cent of the work consisted of the installation of some metal work to be furnished by the city, and the placing of some concrete after the metal was in place. Somebody blundered, and the metal was not delivered for 18 months after the contractor was ready to receive it. In the meantime the contractor had to keep his plant in place, maintain watchmen, pumps and a force to look after the plant, besides keeping his bond in force, losing the interest on the retained percentage amounting to half a million dollars and sundry other items. Eventually he finished the job just within contract time.

The only satisfaction he could get was sympathy. Everybody was sorry, but his work wasn't supposed to be done so quickly. Is that contractor entitled to damages because of that delay?

A gain prevented is a loss sustained, and a contractor is certainly entitled to damages for loss of the use of his plant and money which might well have been earning more money on other work which he was prevented from bidding on because they were unavailable. And the public has already paid in that instance. Not in increased cost of the work, but in inconvenience, for the contract extended through the busiest part of the city, and shaft-structures and the unavoidable street obstructions, all constituted an injury to business impossible to reckon in dollars and cents.

On subway contracts now building in New York, the contractors complain that their working plans have been delayed for months after they were ready for them, thus retarding their progress, and it has been asserted that the serious accidents which occurred in two sections undoubtedly would not have taken place had the steel been erected. On those particular sections the steel plans had been delivered to the contractor only shortly before the accidents—*nearly two years after the contracts had been awarded.*

Those accidents cost the contractors some money—and the public paid too, but their loss was in a measure not reducible to money damage.

A subway contractor stated that because of a 10 months' delay in deciding on a station location and consequent delay in receiving plans, the work was carried over into the period when labor and materials commanded

very much higher prices, and the station cost him about $140,000 more than if it had been done ten months previously, when he should have had the plans. And his is only one of many similar cases.

There is too often a woeful lack of co-operation between engineers and contractors, and this is especially true of engineers on public work. The public's interest is nobody's interest, and work lags and drags in a manner totally absent from construction for railroads or other private utility corporations. One of the most intricate and stupendous pieces of work in connection with the dual subway system in New York was the third tracking of the elevated roads. This was a $15,000,000 job, and it was completed in less than two years. The work was designed by and done under the supervision of the engineers of the Interborough system and has been in operation for a year, while the men are still in some of the trenches under city supervision, which were started long before the third-tracking was begun.

Isn't this a big price for the public to pay? And isn't it an engineering duty—for it can't be called a problem—to reduce this high cost of public work? Some of it—a great deal of it—is caused by too drastic contracts—and a great deal more of it is due to the delay in proceeding with the work, all of which the contractor anticipates and charges for in his bid. Offer inducements to the contractor to complete his work ahead of time, and the saving, if only in the less inconvenience to the public will be enormous.

These are the big things in the dealing between engineers and contractors. They mean large sums of money. The courts are kept busy with litigations over them. And the public pays for the courts.

There are little things in the relations between engineers and contractors that are sometimes annoying, but they don't seriously affect anything but tempers. The man who rejects masonry because the joints deviate a fraction from the specifications, or piles because they are a quarter-inch under size at the butt is either a young engineer in his first job, full of zeal and taking himself a trifle too seriously, or else a pin-head who never becomes a real engineer at all. In the latter case he may cause temporary trouble but is apt soon to be overruled, and in the former, a little experience and judicious talk from the "old man" generally cure his defects.

One of the most amusing instances of the ridiculous extremes to which these sticklers for the specifications will go was that of a young engineer who refused to allow a concrete mixer to operate because a sprocket in a certain part of the machine was stamped No. 8, when, according to the catalogue, it should have been stamped No. 7. Although it was demonstrated that this was a factory error and had not the slightest bearing on the output of the mixer, as shown by the measuring boxes, the engineer stubbornly argued that the mixer was not *theoretically correct*, and work on that mixer had to be suspended until the chief engineer could be reached and reasoned with the young man. It was amusing, but the incident occurred during a rush period, and that particular contractor's opinion of engineers underwent a severe decline.

Nothing that I have said here conveys a new or original thought. The subject has been gone over many times and that's the best indication in the

world that the relations between engineers and contractors are not just what they should be. My connections with many of the largest contractors in the country for the past few years have given me a personal knowledge of their experiences and a close insight into their personal views. Speaking for them in expressing those views, I can't do better than to quote from one who has done more in discussing this same topic and endeavoring to bring about reforms than perhaps any other contractor in the country, Mr. James W. Rollins, president of the Holbrook, Cabot & Rollins Corporation, who, in a recent address before the Engineers' Society of Western Pennsylvania, summed them up in these words: "To give and to take, to do justice on the one hand and good honest work on the other, and settle the difference man to man. Then lawyers will go out of our lives, the courts will be left for trust cases and divorce suits, and the day will come of good-fellowship and regard which should exist between the creators and makers of this modern world—the Engineer and the Contractor."

RECENT TENDENCIES IN INDUSTRIAL BUILDING CONSTRUCTION.

By W. P. Anderson.*

In connection with the work of our company, I became interested to establish if possible something specific regarding the trend toward concrete construction for industrial buildings. To this end a somewhat restricted investigation was undertaken during the past year which covered only the collection of data regarding the floor area, type, and material of industrial buildings erected during the period 1905–15, and part of 1916, within what may be roughly classified as the Middle States.

When the first inquiries were sent out I was not aware of the strenuous efforts being made by the Portland Cement Association to get hold of statistics on the use of concrete in building construction. I soon learned, however, that they had sought information from engineers and architects as well as from contractors, that they had tried to get data regarding buildings used for specific purposes, and had asked for records from the building departments of large cities, but, in the words of their Chief Engineer, "The efforts have been unavailing."

This certainly was not encouraging to one who had already embarked upon a similar investigation. Nevertheless it was pursued, but with a keener realization of the difficulties. Already there had been sent to a selected list of manufacturers in various fields a letter with blank providing space for reports, as indicated by the column headings in Fig. 1. Principal among

YEAR	TOTAL FLOOR AREA IN SQUARE FEET CONSTRUCTED IN EACH YEAR					
	All Wood	Brick Walls		Concrete		Use of Building
		Joist Floors	Mill Construction	Steel Frame	Reinforced Throughout	
1905						
1906						
1907						

FIG. 1.—INQUIRY BLANK.

the industries covered by this inquiry were those represented by the manufacturers of all classes of metal goods, brewers, manufacturers of textiles, paper, leather, boots and shoes. In addition were many others so greatly diversified as not to be readily classified. To this letter there have been received over 1300 replies, many of which indicated that no building had

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TABLE 1.—FLOOR AREAS IN SQUARE FEET AND NUMBER OF NEW BUILDINGS REPORTED FOR EACH YEAR.

Type of Building.	1905.	1906.	1907.	1908.	1909.	1910.
All wood	262,137 22	59,700 12	65,372 9	31,954 7	52,808 13	95,332 18
Concrete reinforced throughout	404,925 8	519,940 10	1,051,598 12	442,295 14	399,091 12	1,552,106 28
Concrete, steel frame	145,385 4	103,515 5	715,657 7	243,850 7	384,731 9	539,575 14
Brick walls, concrete floors	65,629 5	73,817 5	91,874 9	87,600 9	97,399 7	236,966 5
Brick walls, steel frame, concrete floors	1,800 1	204,668 2	9,640 2	15,996 3	92,169 5	29,420 5
Brick, mill construction	1,948,785 35	878,952 26	1,449,075 25	524,032 21	852,857 26	1,299,827 27
Brick walls, joist floors	521,640 19	213,614 11	391,679 16	134,200 14	99,384 11	169,163 13
Brick walls, steel frame	1,117,723 10	700,326 7	895,189 4	271,532 6	282,427 7	434,495 12
Miscellaneous	50,220 4	54,628 4	61,238 5	24,680 2	14,400 1	56,258 2
Totals	4,518,244 108	2,809,160 82	4,731,322 89	1,776,139 83	2,275,266 91	4,413,142 114

Type of Building.	1911.	1912.	1913.	1914.	1915.	1916. 6 to 8 mos.	Totals.
All wood	63,691 13	182,307 14	166,091 8	200,235 11	323,667 17	150,054 12	1,653,348 156
Concrete reinforced throughout	631,360 14	1,152,585 25	1,455,173 22	781,532 15	1,299,358 20	2,076,283 28	11 766,246 198
Concrete, steel frame	475,768 14	822,793 13	496,852 15	70,720 5	423,005 17	593,654 12	5,015,505 122
Brick walls, concrete floors	226,255 8	433,411 14	343,708 10	89,400 3	104,360 6	62,692 9	11,072,885 311
Brick walls, steel frame, concrete floors	326,860 5	249,765 5	214,388 7	68,496 6	548,316 4	176,690 9	1,938,208 54
Brick, mill construction	730,998 24	828,062 33	913,779 26	427,538 17	429,009 19	789,971 32	1,913,111 90
Brick walls, joist floors	227,252 11	233,766 16	415,550 9	184,800 10	143,940 10	347,219 23	3,082,207 163
Brick walls, steel frame	513,363 4	684,547 5	341,013 3	214,984 9	491,958 9	393,014 12	6,340,571 88
Miscellaneous	7,160 3	19,400 4	31,252 3	30,125 4	70,170 8	88,155 8	507,686 48
Totals	3,202,707 96	4,606,636 129	4,377,806 103	2,067,830 80	3,833,783 110	4,677,732 145	43,289,767 1230

NOTE.—The upper figure in each space gives the gross area; the lower figure indicates the number of buildings to which it applies.

been undertaken within the years specified, or gave no information, while others included several buildings. The data utilized in making up the accompanying tables and diagrams were contributed by 370 concerns and cover 1230 buildings varying greatly in size, use and construction, but all used for industrial purposes.

In many cases the returns indicated special features of construction that made it extremely difficult to properly classify the buildings under any of the headings provided on the original data sheet. Thus, for instance, numerous buildings were reported as "brick walls, mill construction," but with steel frame, while others were specified merely as having "brick walls, steel frame," sometimes with wood floors, sometimes with concrete, and usually without statement as to the character of the roof. This was noticeable in the case of one-story buildings.

However, the possibilities of improper grouping were very generally eliminated by careful analysis of the returns and amplification of the classifications as indicated in the complete report shown in Table No. 1.

TABLE 2.—FLOOR AREAS OF FOUR PRINCIPAL TYPES OF CONSTRUCTION IN THE CASE OF NEW BUILDINGS REPORTED FOR EACH YEAR, 1905–1916.

Year.	All Wood.	Mill Construction.	Concrete Construction.	Brick Walls, Steel Frame.
1905	262,137	2,470,425	617,739	1,117,723
1906	59,700	1,092,566	901,940	700 326
1907	65 372	1 840,754	1,868,769	895,189
1908	31,954	658.232	789 741	271 532
1909	52,808	952,241	974 390	282,427
1910	95,332	1,468,990	2,358,067	434,495
1911	63,691	958,250	1,660,243	513,363
1912	182,308	1,061,828	2,658,554	684,547
1913	166,091	1,329,329	2,500,121	341,013
1914	200,235	612,338	1 010,153	214,984
1915	323,667	572,949	2,375,039	491,958
1916	225,081	1,705,785	4,363,975	585,521
Total	1,728,376	14,723,687	22,078,731	6,533,078

Such diversified grouping manifestly reduced to a comparatively small number the buildings which fell within each subdivision. Nevertheless the returns are presented in detail so that the subsequent analysis may be better understood. The figures giving buildings and floor areas finished in 1916 cover a somewhat uncertain period of 6 to 8 months in the year, for the reports were made during the late summer. They are particularly impressive when compared with those for previous full yearly periods.

In making up Table 2 and plotting the curves in Fig. 2, the total for 1916 has been estimated in each case on the conservative basis of being 50 per cent more than the total reported for the period that manifestly averages less than 8 months.

It is obvious that so far as the general character of construction is concerned and for purposes of comparison it is justifiable to segregate the returns in more comprehensive groups designated as All Wood, Mill Construction,

Concrete Construction and Brick Walls, Steel Frame. As so arranged and shown in Table 2, which gives the aggregates for each year of the entire period, the first three divisions are reasonably definite in their limits, but the fourth —covering as it doubtless does many one-story buildings—leaves one in doubt regarding the material of the floor and the roof. The unclassified buildings listed in Table 1 as "Miscellaneous" have been omitted.

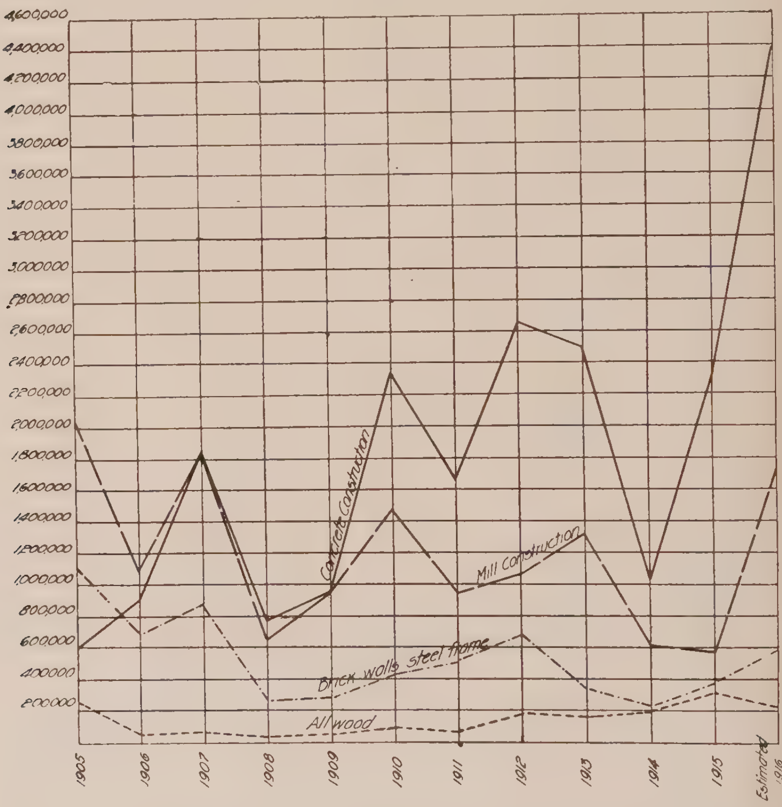

FIG. 2.—TOTAL FLOOR AREA OF FOUR PRINCIPAL TYPES OF CONSTRUCTION IN THE CASE OF NEW BUILDINGS REPORTED EACH YEAR, 1905–1916.

On the same basis of segregation the returns year by year are graphically presented in Fig. 2 for the four major classifications. Of course the abnormal disturbances during 1914–16 account for the great fluctuations in that period. But even up to and including 1913 the advance of concrete construction and the relative decadence of other types was distinctly noticeable. Even in 1913 when for a single year the former type dropped while that of mill construction increased, the former exceeded the latter by 88 per cent; in 1916

the excess amounted to 156 per cent. The comparative growth is most vividly shown, however, by comparing the periods 1905–10 and 1911–17. In the former the returns cover 7,014,218 sq. ft. of mill construction, and only 5,152,579 sq. ft. of concrete construction. But in the latter period the area of concrete construction jumped 329 per cent to 16,926,152, while the mill

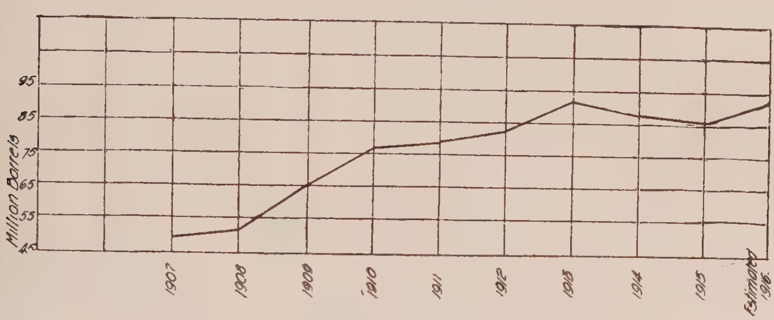

FIG. 3.—PRODUCTION OF CEMENT, 1907–1916.

construction showed a bare increase of about 10 per cent to 7,709,469. The estimated area for 1916 was used in making all of these comparisons.

It is interesting to compare the curves in Fig. 2 with one (Fig. 3) showing the annual production—not the consumption—of cement during 1907–16 as reported by the Portland Cement Association.

UNIT CONSTRUCTION IN CONCRETE.

By John E. Conzelman.*

We are facing a crucial period in the development of concrete construction. During the past twenty years we have been so occupied in developing and constructing that we have had little time for sober second thought and investigation. Fortunately the last two or three years have witnessed a change and most of us have paused to analyze the results thus far accomplished.

We realize now more fully than we used to that concrete is not permanent unless it has been made of carefully selected materials which have been properly proportioned and thoroughly mixed for a reasonable length of time. We have known that the water content exerts a marked influence on the strength and permanence of concrete, and most of us have been guilty of varying this factor to suit the requirements of our spouting and conveying systems. It is well for the art that we have reached this transitional stage and are profiting by our past experiences. Practical experience combined with the analyses and investigations that have been made have not only pointed out the defects but have fortunately shown the causes and indicated, at least, to a considerable extent, the remedies.

In this period of reformation there are few engineers or constructors who are not feeling the spirit of the times, and who are not applying what they have learned to the improving of the product. This is a healthy condition for any industry and one that will lead to large results.

We have not only learned that we have been neglectful of these precautions that affect the permanence and uniformity of the concrete produced, but we have further realized that these same precautions would have resulted in a great increase in the strength of the concrete. We have not been using the materials to the best advantage and this has acted to retard progress. When we use mixers that will work the materials together instead of merely agitating them we will have gone a long way in the right direction. Proper selection of aggregates and thorough mixing should result in concrete of such reliable and uniform strength that a saving of 30 per cent of materials could be accomplished. This statement is, of course, predicated on the assumption that the concrete will be properly protected during the curing period.

It may be said, therefore, that careful selection of aggregates, skilful proportioning (including water), long mixing and proper handling and placing, combined with reasonable protection after the concrete is in the forms, are the basis of successful, economical and permanent construction.

Unit methods offer superior opportunities to secure the results outlined. Concrete is mixed and placed under what may be called factory conditions

* Civil Engineer, St. Louis, Mo.

and because it is possible, and necessary for economical operations, to pour each day there is no tendency to rush concreting at certain periods; system and orderly procedure result in a better product in this case as elsewhere. Those who are familiar with both constructions know that concrete produced under unit methods is a greatly superior product.

As a result of these advantages the use of unit methods has increased more extensively than is generally known. The construction has been extensively used for practically all structures ordinarily built of reinforced

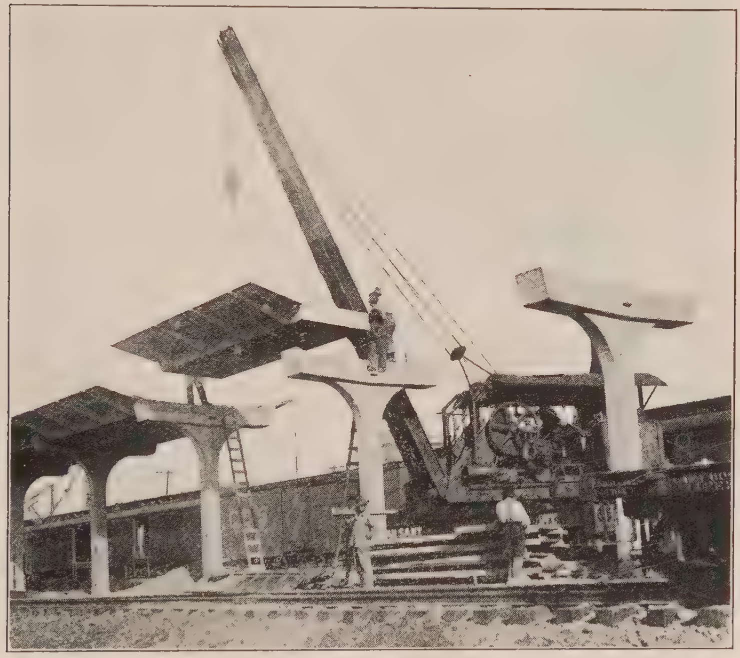

FIG. 1.—PLACING ROOF SLABS ON RAILWAY UMBRELLA SHEDS, SAN FRANCISCO, CAL.

concrete, and in addition to this has opened up entirely new fields for the application of this material.

RAILROADS USE UNIT METHODS.

Railroad work offers a wonderful field for the development of the unit method. If it were possible to standardize such structures as engine houses, freight sheds, snow sheds, train sheds and small stations, the construction of these by the unit method on a factory basis at central locations would effect economies that would be surprising. This method would also insure permanent and reliable structures.

FIG. 2.—ROUNDHOUSE FOR SANTA FÉ R. R., RIVERBANK, CAL.

In my judgment such methods would produce a saving of 30 or 40 per cent of the concrete that would be required if the construction were carried out in the usual way. There is no reason why such standardized buildings cannot be carried in stock and shipped out as required. The erection could be done in a surprisingly quick time. What is true of railroad structures applies with equal force to commercial buildings, manufacturing centers and housing developments.

Unit methods have been developed by the railroad companies mainly

FIG. 3.—SUBWAY ENTRANCE TO ARCADE DEPOT, LOS ANGELES, CAL.

FIG. 4.—DOWNSTREAM SIDE OF THE POWER HOUSE, CEDARS RAPIDS MFG. & POWER CO., CANADA.

in the direction of heavy slabs for bridge and trestle work, and at various times engineers have devised unit methods in carrying out particular problems. This is especially true of the construction of sea walls, caissons and similar structures.

The pursuit of the unit idea to its logical development has been mainly carried out by two or three individuals and companies. These parties have expended a great deal of time and a large amount of money in working out their ideas to a practical basis, and some have felt it necessary to take out patents for the purpose of protecting their investments. For example, the company with which the speaker is connected has spent perhaps $300,000

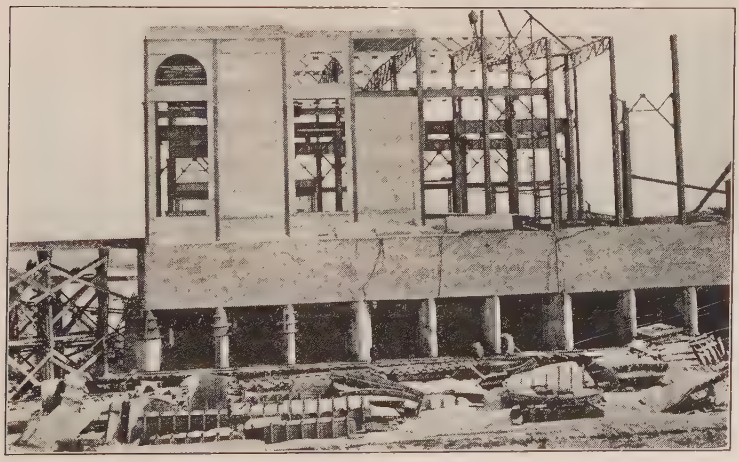

FIG. 5.—CONSTRUCTION VIEW OF THE POWER HOUSE, CEDARS RAPIDS MFG. & POWER CO., CANADA.

on the development of the unit idea. Other individuals have spent large amounts. When it is remembered that most of this development work was carried out under adverse criticism from practically all supposed authorities is it not but just that the work of these companies should be recognized, and that their rights to the methods developed should be respected by all?

When the time comes, as it soon will, when units are made in permanent factories by improved methods of manufacture and curing, a perfectly

FIG. 6.—CASTING YARD OF UMBRELLA RAILWAY STATION SHEDS SHOWING ROOF SLAB FORMS.

reliable structural material will be obtained, and perhaps 50 per cent of the concrete that is now used will be saved.

I wish to call special attention to the construction used for the power house for the Cedars Rapids Manufacturing and Power Co. in Canada. The combination of structural steel and concrete as used there produced a building of low cost and yet one that should be practically permanent. With a very little additional expense a hollow wall construction was secured which

is a desirable feature in that climate. The possibilities of this combination for the construction of piers, power houses and factory buildings of permanent character make a very strong appeal to the imagination.

WORKINGMEN'S HOMES.

The economic construction of sanitary, permanent and fire-resisting homes by unit methods for workingmen and others is a demonstrated possibility. In no other field of constructional endeavor are the possibilities of economic results of such universal advantage.

The construction of homes, especially the low-priced house, has not received the intelligent, thoughtful and scientific planning that is used for larger developments such as manufacturing plants or office buildings. In this case, also, it has been found impossible to use scientific construction methods on account of the limited costs. In this case, as in most others, the poorer man who needs it most is prohibited by circumstances from

FIG. 7.—CLASSING SHEDS FOR THE GALVESTON COTTON COMPRESS & WAREHOUSE CO.

utilizing the best methods and the efficient organizations devoted to building construction.

The construction of workingmen's homes has in many cities been left to the speculative builder whose aim has been to produce the most for the money with little regard to quality. The result is poor construction and continual repairs—repairs often being justified by the statement that the laboring man, or mechanic, likes to have something to do around the house. This, of course, is a false policy.

At the present time in practically all of our manufacturing centers there is a great lack of housing accommodations, skilled workingmen receiving high wages find it impossible to secure satisfactory or even decent accommodations. This condition can only result in a lack of effectiveness.

The problem has, however, an entirely different aspect when we come to the consideration of workingmen's colonies. In this case if the development comprises a sufficient number of houses, the total cost becomes an item sufficient to attract services of the highest class and justifies the use of equipment on a large scale. This step at once brings to the construction

of workingmen's homes the same architectural, engineering and constructional skill ordinarily used in the construction of large building operations.

It is, in fact, the one method by which the greatest economy can be obtained in the construction of individual homes. It is, also, the one method by which the ornamentation of one house can be omitted and attractiveness obtained by the arrangement of entire blocks. The same principles of economy and rapid construction apply in this field as in the railroad construction field heretofore mentioned.

FIG. 8.—POWER HOUSE OF THE POSTEX COTTON MILLS CO., SHOWING ERECTION OF A 42-FT. TRUSS, POST, TEXAS.

The standardization of parts and construction by factory methods mean the same economies in building construction as have been obtained in manufacturing operations of all kinds. The possibilities of constructing houses by the use of standardized units, even when the development is not carried out on a large scale, has been demonstrated. Mr. Grosvenor Atterbury of New York has constructed 16 or 18 houses with standard sections in which the walls, floors and roofs are of reinforced concrete. Fourteen of these buildings were constructed under the direction of the Sage Foundation at Forest Hills, Long Island.

FIG. 9.—INTERIOR MILL BUILDING OF THE POSTEX MILLS CO., POST, TEXAS.

These houses which have now been in use for several years, and which are rented to a high-class of tenants, have proved entirely satisfactory, and I understand arrangements are now being made to erect a great many more. In these houses the walls are constructed of hollow standardized sections or units and act as supports to the unit concrete floors.

The speaker took up the question of the study of workingmen's homes about eight months ago and in this time has developed several methods for

FIG. 10.—CAR BARN OF THE PHILADELPHIA RAPID TRANSIT CO., SHOWING CONSTRUCTION OF REINFORCED-CONCRETE SKYLIGHTS.

the economical and rapid construction of workingmen's colonies. None of these colonies have as yet been constructed, but tenders have been submitted on the construction of approximately 4000 houses. With the experience gained in the construction of a large number of manufacturing plants and other structures by unit methods the costs of these houses are figured with reasonable exactness. The estimates show that if the construction of these houses can be carried on as a continuing operation that the cost of the reinforced concrete portions can be kept down to a surprisingly low figure.

It is my belief that under such conditions these houses, which will be sanitary, fireproof and permanent, consisting of reinforced-concrete walls, floors and roofs, can be constructed at a cost to compete successfully with the usual construction as ordinarily carried out. These conclusions are the results of investigations made in reporting on various houses prospects.

DOES CONCRETE CONSTRUCTION REDUCE VIBRATION?

By Morton C. Tuttle.*

The following expressions of opinion and reports of experience which have been received in connection with the Aberthaw Investigation of the Effects of Vibration in Structures are presented somewhat in the form of a discussion in the hope that they will encourage further expression. Manifestly there is no attempt on the part of the writer to draw final conclusions; such effort will be reserved until additional information has been collected and the experimental portion of the investigation is practically completed.

The first step in the investigation was to send letters to a selected list of manufacturers and engineers asking among other questions whether they had been able to trace any effects to the vibration of buildings, and if so in what types of construction. Out of nearly 1150 replies, almost 400 contained matter of a really helpful or suggestive nature. It is from this group that the following quotations have been selected as presenting various points of view regarding the relation of concrete construction to vibration in structures.

The obvious inference that such construction conduces to stability and to the elimination of vibration is voiced by many in such very general terms as "my experience has been that all brick buildings vibrate more or less and the one remedy I can suggest would be to build our future factories of heavy reinforced concrete. This seems to be the prevailing opinion of our best engineers."

One writer simply says that he has "much faith in reinforced and suitable foundations," while another who has had no experience with concrete constructed buildings asserts, "there is no doubt in my mind, however, but that a rigid building will add considerably to the good running of the machinery," and a third, whose buildings are of slow-burning mill construction and reinforced concrete states, "of course it goes without saying that the amount of vibration with reinforced-concrete buildings is considerably less."

Speaking of their experience in buildings of light mill construction a confectionery concern concludes that "steel and concrete construction would be more desirable in such cases," while a firm of silk manufacturers express their preference for "buildings of reinforced concrete, flat slab, in which type of building we believe vibration is reduced to a minimum."

That people build with faith in concrete is shown by this quotation: "During the past year we have erected a new reinforced-concrete building with slab floors 10 in. thick, and we feel very sure that the matter of vibration will be eliminated entirely."

* Aberthaw Construction Co., Boston, Mass.

It will be recognized that such expressions carry comparatively little weight of experience—in fact most of them are from letters of a very general nature which convey no authoritative information.

In a slightly more restricted class may be grouped statements based upon individual experience somewhat more specific in their nature. Very concise is this from a large printing company: "In our new building vibration is hardly noticeable. The construction is steel and reinforced concrete." The experience with and advantages of reinforced-concrete construction are thus expressed by another correspondent: "We have so far found the building without any vibration whatsoever which we believe is of great benefit owing to the fact that many sensitive automatic machines are used by us for making watch cases." From a large pulp and paper manufacturing concern comes this statement: "We find the combination of steel and concrete to be the only satisfactory construction for our purpose as the mass and rigidity of such construction insures practical freedom from vibration."

A well-known builder of reinforced-concrete structures says: "My observation of the effect of vibration on concrete is that if the concrete is good it withstands vibration better than any other material used in the building business. In general there is much less vibration in concrete buildings than in any others."

A shoe manufacturer speaking of the necessity that some of their machines should run absolutely true, and reporting experience in buildings of wood construction where even with the machines braced from the floor to the ceiling the vibration is so great that the operator has great difficulty in doing much or good work, sums up his experience in these words: "I find the same trouble in brick and wood buildings, but in concrete buildings bracing is not necessary and all machines run truer." A typical case of contrast between structures is expressed in these words: "We are troubled in one of our buildings, which is of mill construction with floors made of 2 x 4's placed edgewise and covered with maple flooring. The vibration of shafting on the lower floor is transmitted to the next floor above. This has proved objectionable to the workmen. Our other buildings are of concrete construction with reinforced floors and we have no trouble in this regard."

Definite statements regarding the quality and quantity of output in concrete constructed buildings are most suggestive of the real value of such construction in the elimination of vibration. A very specific statement to this effect is found in the following: "We have found that we can produce our goods cheaper since we placed the looms in the new concrete building due to the absence of vibration." A motor-car manufacturer, speaking of timber construction, says: "Gear-cutting machines or grinders cannot be as successfully used here as in a building of reinforced concrete. Fully twice as accurate work can be done on a firm foundation."

"All of our machines and especially looms," says a textile manufacturer, "work better on solid floors—therefore the remedy we believe to be in concrete construction." In a case where wooden floors have been replaced with concrete in a brick wall building the writer says: "I am safe in saying

our percentage of breakage has been less and output per machine better with less annoyance." As presenting some conflict of opinion is the following from a leading mill engineer and architect. Speaking of the work of an investigator who has made the claim that machinery could be run faster in reinforced concrete than in regular mill construction buildings on account of the lack of vibration, he says: "But I am inclined to think that the limitation is found in the machine itself and not in the building."

The investigation reveals very clearly the necessity of understanding all of the conditions if judgment is to be passed as to the causes or effects. Of course the type of machine and the method of driving have much to do with causes of vibration. One concern says: "We drive all our machine tools with individual motors and have developed a system of individual motor drive for existing machine tools which have been designed for the old-fashioned line shaft drive. With concrete floors and foundations and individual motor drive, all vibration is practically eliminated and the form of construction in buildings no longer enters into the problem except in buildings of more than one story. Even in buildings of several floors with machine tools on each floor the question of vibration is very much minimized by individual motors for each machine tool and the elimination of line shafts."

A manufacturer of rubber goods sends the report of a very careful investigation made to determine how the somewhat localized but very disturbing vibration caused by heavy mills in a reinforced-concrete building could be eliminated. He sums up his opinion in these words: "After our experience, I firmly believe that the floors in so far as possible, of concrete buildings should be independent of the outer wall. I furthermore believe it would be a good plan to set all heavy machinery on piers or foundations which are also independent of the floors of the building. This probably would only be possible in basements or on first floors and would not be practical in the upper stories."

Many reports have been received regarding conditions in one-story shops, practically all of which, as would be expected, indicate the absence of vibration. Frequently reference is made to substantial foundations or to heavy concrete ground floors as responsible for this satisfactory condition. In general terms the expressions are about as follows: "All of our buildings are one-story structures, mostly brick walls, mill constructed. All floors with a few exceptions are concrete—a few of the buildings are concrete throughout. We have noticed no excessive vibration nor have we had any trouble or noticed any effect on the product of our men from this cause." The manufacturer of a line of high-grade machine tools, speaking of the fact that "in machine tool manufacture vibration usually causes rough and inaccurate work which means extra hand work such as scraping," say of their own plant: "Our entire building is of concrete and all our heavy machinery is on the ground floor. As regards machine operation we have very little trouble which we can trace to our buildings. We have had difficulty, however, in taking fine measurements where very sensitive indicating mechanisms both mechanical and optical are used."

But all reports are by no means favorable to reinforced-concrete construction. In many cases the reason is evident in the fact that the building is not suited for the processes carried on within it. Frequently the unexpected happens, due quite largely to the synchronizing or getting in step of a group of similar machines. One mill architect speaking of a printing plant says: "They have their presses in a concrete building and I understand the vibration is quite noticeable." He also states: "I have noticed that the same stamping machine in a mill-constructed building causes much more vibration than one installed in a reinforced-concrete building." Another engineering concern says: "We have built some reinforced-concrete buildings, flat slab design, for concerns using stamping and similar machinery for cutting shoe soles and parts. There is a good deal of vibration but no harmful results have been observed."

Some of the most distressing conditions are reported in connection with printing plants. Here is one relating to a reinforced-concrete building five stories high and 300 ft. long which was to have had its numerous presses on the second floor, but the owners "were so impressed with the stiffness of the structure that the presses were finally put on the top floor, all parallel and running lengthwise of the building. The vibrations were said to be alarming at times, but were satisfactorily overcome by turning half at right angles, with the result that nothing further on the subject has been heard by the engineers."

In pleasing contrast to the preceding experience is this regarding a large printing company who put up a reinforced-concrete building: "Previous to their occupancy of this building they had been located in an old brick and wood building. They find that they can now run their presses 20 per cent faster than in the old building. They also state that the reduction in vibration has increased the comfort and the efficiency of their employees and the output of their machines."

The whole story regarding suitable construction is summed up in these words from a paper manufacturer: "We have both frame and concrete and steel construction, and have found that where we had trouble on account of vibration that was so strong as to interfere with the operation of the machinery it was due to light construction in the frame as well as in the concrete and steel. Whenever reconstruction was necessary we built much heavier and the required weight of material was put into foundations and otherwise. In most cases this solved the problem." Another paper manufacturer having steel and concrete buildings says: "They are practically rigid but there is some vibration from the shaking motion of our diaphragm screens. We have never seen any bad results from this vibration although it has been practically continuous for ten years."

Very emphatic is this from a manufacturer of farm implements: "We have had a great deal of difficulty in our concrete buildings due to the vibration from certain classes of machinery. We have in mind particularly the sheet metal working presses and shears which are placed on the fourth floor of one of our new concrete buildings. The vibration from these seems to be just the right amplitude to produce a disagreeable and even dangerous

effect throughout the entire building. We have finally put large rubber pads underneath all of these places and we think that this has largely overcome the difficulty, although in shearing heavy plate there is still a very appreciable vibration."

A valuable contribution to the discussion is made by a concern intimately connected with concrete construction, the chief engineer of which says: "I have been in a great many reinforced-concrete buildings in which high-speed machinery has been used, and also in many textile mills, and while I was not examining these buildings with the idea of determining the type of construction to produce the minimum vibration, except in the case of the textile mills, I did note that where the so-called flat slab construction was used the vibration was heaviest, and that there also was considerable vibration in buildings composed of beams and girders with short span slabs. The vibration seemed to be reduced to a minimum on jobs where the floor slabs were deepest, and on this account we ourselves in designing several textile mills adopted a floor slab composed of terra-cotta tile and concrete joists, the object being to secure as deep a slab as possible without at the same time increasing the cost and weight to any material degree."

An expert in concrete construction who says that his observations have been only casual and that his conclusions on the same are in consequence only general, states:

"1st. It is very difficult to set up harmonious vibrations in a concrete structure.

"2d. Localized vibrations are apparent in most concrete structures.

"Heavy machinery, like printing presses, sometimes vibrate the whole building to such an extent that the movement is noticeable without instruments. Heavy cutting machines appear to vibrate floors of buildings only locally."

This critical statement from an expert engineer is worthy of especial attention: "Since the development of reinforced concrete, with its inherent elasticity, the writer has used and recommended the use of that type of masonry. Prior thereto have used the 'hoop iron bond' and brass, iron, wood and copper cramps.

"Have made a number of observations on tall structures; and have not found one that did not 'come and go' under a 'whole sail breeze.' Have also noted the swing due to temperature and earth tremors. The result of all of it is that the writer can only say that any structure should not be rigid. At present the most flexible masonry we have is reinforced concrete.

"As a matter of history, the U. S. Government building in San Francisco, which went through the earthquake, is most thoroughly laced with strap iron and cramped; a fact to which the writer can certify, but which has probably been forgotten."

The difficulties attendant upon an investigation of vibration and the reasons why the whole matter is still in a controversial stage are well expressed in the following from a consulting engineer: "The cause and effect of vibration in buildings are so uncertain, unless especially studied, that I should think that the average engineer would be able to give a little attention to

it and arrive at a conclusion that would be worth mentioning. So far as my own experience goes I believe that the monolithic structure is the right answer. I have found very little vibration in buildings of that type, whereas there was considerable vibration in buildings of other types containing the same kind of machinery and carrying on the same processes."

Many more suggestive quotations from our correspondence might be presented, but the writer trusts that these will be sufficient to draw out discussion. He is particularly anxious to obtain authentic data which will assist in arriving at definite conclusions regarding the exact status of reinforced-concrete construction in its relation to vibration.

DISCUSSION.

Prof. W. K. Hatt.—It is a curious thing that vibrations which seem large to a person in a room are so small that they can hardly be measured. That was the case with the Masonic Temple in Indianapolis, where they used to drill on the roof below which was a big audience hall. The marching of those men on the roof produced such vibrations that it loosened the electric lights in some of the rooms above and gave a general sense of insecurity. They got alarmed and had measurements made of the vibrations going on overhead, and the vibration was found to be actually so small that it was scarcely able to detect it. **Professor Hatt.**

Mr. Richard L. Humphrey.—A building in Philadelphia was built in two halves, one of steel, the other of concrete. Standing in the steel structure when the press was running you could feel distinctly the vibration; standing in the concrete structure when a similar press was running, you could not feel any vibration or tremor. **Mr. Humphrey.**

Mr. A. B. Cohen.—The Delaware, Lackawanna & Western R. R. has just completed at South Orange, N. J., an elevated structure of the cantilever flat slab type, 425 ft. long and 80 ft. wide, with panels 18 x 20 ft. The depth of the slab is 1 ft. 8 in. The station is built underneath this elevated structure and the rigidity of the structure is remarkable. There is very little noise underneath, and two trains passing over it at great speed will produce very little of the noise usually found in elevated structures of the steel type. **Mr. Cohen.**

Mr. W. A. Collings.—In Kansas City we were recently called in to look at a large printing plant, 6 stories high, in which there were presses on the third, fourth, fifth and sixth stories. There was so much vibration in that building that people threatened to get out. We tried to change the presses around so that they did not synchronize, which helped some, but finally we braced the columns so that the vibration was reduced to a minimum. In another printing plant we put in between two columns at the window sills a solid block of concrete, about 3 ft. high, to serve as a wind brace. That has absolutely stopped all vibration in that 8-story building, and while the stresses are very much heavier, there is not the slightest vibration. **Mr. Collings.**

BUILDING CODES FOR SMALL TOWNS.

By Ernest McCullough.*

Before going into the subject assigned, the writer wishes to present for consideration the following from the 1915 edition of the building code recommended by the National Board of Fire Underwriters:

> ### Foreword.
>
> Since the publication in 1905 of the first issue of this Building Code, it has passed through three editions and over 20,000 copies have been distributed to the public.
>
> The "Foreword" of the original edition contained the following paragraphs:
>
> "In the belief that safe and good construction of buildings should be universally recognized as of the utmost importance, this Building Code, prepared and recommended by the undersigned Committee, is based on broad principles which have been sufficiently amplified to provide for varying local conditions. . . . .
>
> Thousands of human lives and millions of dollars' worth of property have been sacrificed by the criminal folly of erecting unsafe or defective buildings. So long as those in authority permit such buildings to be erected, neither life nor property can be safe. A remedy safeguarding both may be found in this book. The vital importance of its principles should arouse municipal authorities everywhere to a realizing sense of their duty and to the grave responsibility that rests upon them to enact and enforce adequate building laws for the protection of life and property."
>
> The Committee considers those statements as true today as when first presented, and believes they apply to this revised edition with even greater force than to the one for which they were originally written. In the period intervening between the two, the art of building construction advanced with remarkable rapidity. Materials employed and methods of use have changed radically in certain directions, making a revision of the Code a necessity. The need for occasional revision was foreseen and indicated when the Code was first written, and will of course be required from time to time to meet changing conditions.
>
> The Committee in presenting this completely revised edition, believes it offers to the public a safe, practical, conservative building ordinance which represents the best engineering practice of the day.
>
> The revised edition contains three new features. One is the use of numerous notes scattered through the text, serving either as recommendations, cautions, or explanation. The second is the introduction of cuts illustrating details of construction, and the third is the use of fre-

* Consulting Engineer, Chicago, Ill.

quent cross references in the text which will aid in quickly finding allied subjects. The object of these changes is to make the Code a guide or text-book for the use of Commissions engaged in drafting building ordinances, and is designed to cover all the essential features of construction which such a Commission would be likely to discuss. The Committee recognizes it is impracticable to draft a standard code suitable for adoption without change by cities in all parts of the country. Local conditions must necessarily govern regulations covering sanitation, morals, or other important subjects which may be deemed desirable to include in the building code of a city but which are only treated incidentally here.

Attention is called to the "Suggested Building Ordinance for Small Towns and Villages," also issued by the National Board of Fire Underwriters for the use of municipalities too small to require an ordinance as comprehensive as this one.

The Committee welcomes correspondence from cities which are drafting new Codes or revising old ones, and is prepared to furnish assistance gratis in such work so far as it can consistently.

The Committee desires to express its appreciation of the assistance rendered it by helpful suggestions, and valuable criticisms willingly given by many structural engineers and architects whose wide practical experience made their comments most valuable.

In 1916 the National Board of Fire Underwriters issued a pamphlet on dwelling houses. The writer feels under no necessity for apologizing in presenting here the Foreword of this pamphlet:

Foreword.

The National Board of Fire Underwriters has received many requests for construction specifications which will properly protect dwelling houses against fire. This pamphlet has been prepared in response to such inquiries, and is addressed directly to owners of dwellings and to carpenters and builders who erect them.

The majority of dwellings are outside the control of building ordinances, and those within the jurisdiction of such laws usually have but few restrictions; hence the field for use of this information is broad.

The few specifications governing dwelling house construction in a Building Code are usually so scattered through the ordinance it is difficult for a person unfamiliar with such laws to ascertain exactly what is required. That fact further justifies codification of these well-established principles for the protection of such structures.

The pamphlet is distinctly educational in character, since naturally there is no authority to enforce its provisions. Many of the more important suggestions were extracted bodily from our Recommended Building Code, and still retain the imperative form of expression "shall" as indicating in our opinion the strict necessity for the requirements.

Although other parts may be expressed less positively, it should not be inferred that they are unimportant.

The principal idea in the preparation of the pamphlet has been to indicate so plainly the structural features necessary to make any house reasonably fire-resistive, that even a layman could understand them. It is hoped that home builders may be sufficiently impressed with the logic of the requirements to voluntarily adopt them.

Aside from the personal satisfaction and peace of mind resulting from owning a home that is known to be as safe as care and forethought can make it, there are other benefits in prospect. Underwriters are considering plans for a scientific classification of cities according to their fire hazard and a grading of buildings based upon their location and construction. When this is accomplished, buildings of good construction will receive a deserved recognition which has hitherto been impossible.

The endeavor has been to recommend the most efficient and practical methods of fire protection, to warn against unsafe construction customs, and to urge best structural practice generally.

That the pamphlet may fulfil these functions satisfactorily, the Committee asks coöperation from those receiving it in making it as reliable and beneficial as possible to the general public.

With this end in view the readers of the pamphlet are requested to send the Committee any suggestions or criticisms which they think would contribute to its betterment. All such communications will be given careful consideration in the preparation of a new edition.

The three books: "A Suggested Building Ordinance for Small Towns and Villages," "Building Code Recommended by The National Board of Fire Underwriters," "Dwelling Houses: A Code of Suggestions for Construction and Fire Protection," may be obtained on request from Ira H. Woolson, Consulting Engineer, National Board of Fire Underwriters, 76 William Street, New York City.

The writer believes these books are unequaled as text-books on building construction. They should be so used in all schools of architecture and engineering. They should be used for reference in the offices of all city officials having anything to do with granting building permits. In the absence of detailed building codes, when reference is made to "approved engineering practice," it should be stated that these books contain a complete definition of such requirement.

A building code is required for one, or for several reasons:

1. To provide for protection against fire and reduce insurance rates.
2. To safeguard life against risk of fire.
3. To safeguard health.
4. To render working conditions endurable in factories.
5. To standardize methods of building construction, thereby lessening costs in a legitimate way.
6. To insure good, safe construction and check dishonest builders.
7. To give authority for adequate control at all times over new buildings and the repair and reconstruction of old buildings.

In the history of every town a time comes when it is believed that a building code is necessary. The usual procedure is to appoint a building code commission. The benefits to be obtained are nebulous and the average city father hates to pay money for dreams. The altruistic sense of the community is appealed to, and men are found willing to perform a public service without pay. These men are engineers, architects and builders. A lawyer is usually attached and he is paid. Many years of experience have shown that legal advice to be worth anything costs money. The clerks and stenographers are also paid.

With such a commission the paid employees do most of the work. Copies of codes are obtained from other cities and a scissors-and-paste-pot code is shortly compiled. The commission then begins sitting. Special pleaders for materials and appliances gather and each attempts to secure special consideration. When the code commission is through and the code recommended for adoption, the city council committee has a series of hearings and the same fight is staged by men not satisfied with the action of the code commission. Then junketing trips begin. Codes prepared in this manner often require years for adoption. After the building material men are through the labor unions and contractors are heard from. Lastly come the real estate speculators who express a great interest in the poor man whom they claim cannot own his home if the cost will be increased. Good building is economical in the end, but the first cost may be slightly greater than the first cost of unregulated construction. The writer has served on a number of building code commissions and can speak feelingly.

In the average small town building ordinances are seldom properly enforced. The services of a competent man can seldom be had to enforce the laws. The work is generally given to a practical (so-called) building mechanic whose ignorance imposes burdensome interpretations on some sections of the law; and whose need is often the cause of violations by men who would rather spend money to bribe a public official than spend it to get good work.

It is not an uncommon experience to permit wealthy men to build as they please when they threaten not to build if compelled to follow the law strictly. A few occurrences of this sort and the building ordinance becomes a dead thing.

Now just what kind of regulations should be adopted for small towns? To what extent should the framers of a building code go in preparing such a code? Shall there be an endeavor to secure as good construction as will be required in a very large city and thereby cause prospective manufacturers to locate elsewhere, or will the requirements be as light as possible in order to help the local chamber of commerce in its work of trying to make an industrial center of the place? Shall the regulations call for absolute safety to life or shall they be such as to merely slightly reduce the fire loss and slightly reduce the insurance rates? The writer believes that this vexed question has been very nicely handled by the National Board of Fire Underwriters in the building ordinance for small towns and villages. It will be difficult to

improve upon that code except to add some of the suggestions in the bulletin issued during 1916 on frame buildings.

This short ordinance of the National Board of Fire Underwriters merely lays down the fundamental requirements for good building from the standpoint of the insurance man. It is not a specification ordinance, and the design of buildings is covered in the one general clause "That the design shall conform to the best engineering principles." To take up this building code in detail and discuss it would render necessary the reprinting of the entire code, for it only fills thirty-two pages of a small size, in good type with plenty of leading between the lines. First, a fire limit is established and the jurisdiction defined, then follow specifications for thickness of walls and for common methods, sufficient to illustrate the difference between good construction and poor construction. The writer believes that in any town having no building ordinance this building ordinance should be adopted in its entirety.

An ordinance such as this will not be in use very long before certain other matters will come up for decision. It will be time then to pass additional ordinances which will have the effect of some day resulting in the evolution of the small ordinance into a complete ordinance fit for a city of any size.

The writer believes that the checking of plans for structural design should not be done at the expense of the town. He believes that to every set of plans there should be attached a statement by the owner, under oath, to the effect that he had requested the designer of the building to follow the building code strictly, and in places where the building code was not specific he was to follow the best engineering principles. Following this statement should be a statement, under oath, by the designer of the building, be he engineer or architect, to the effect that he had complied in the design of the building with all the requirements of the building code, and that where the building code was not specific in its design and requirements, he had followed the best engineering practice as set forth in the books first referred to, and state the date of the edition of the respective books. A complete copy of his detailed computation should be attached and made a part of the records. The plans and computations should be retained in the files of the building department for several years, and if at any time it develops that the parties had made a misstatement, they can be punished. The contractor should be made to take an oath that he will follow the specifications and requirements of the building code in every particular. When the building is completed he shall make a statement under oath that he did so and this statement should be filed in the office of the building commissioner before a final certificate can be issued to him by the architect.

Frankly, the writer has had so much experience with building ordinances in small towns that he is dubious about any great measure of success being achieved in securing the adoption of building ordinances. He believes the time is rapidly approaching when every state will have a building ordinance operative in every village, town and city in the state. Only in this way can very much good be accomplished, and we should work to secure the adoption of proper state building codes rather than waste much effort on the adoption of building codes in small towns.

Safety to property and person should be secured to as great an extent as possible. Every city council meets at least once a month, while some of them meet oftener. Legislatures meet but once in two years, so whenever the legislature meets an attempt should be made to secure the passage of a state-wide building code. If the legislature adjourns without passing such a code, then every man who is interested in proper building should take it upon himself to secure the adoption of the underwriters' building code in the town in which he lives. If all of the towns and cities in the country adopt building codes, it will not be very long before state legislatures will act to provide uniform state codes. The State of Wisconsin has a building code which is not a specification code and the State of Ohio has a building code which the writer understands is a specification code. The State of Illinois had a building commission in existence for some time which prepared a specification code and presented it to the state legislature two years ago. It was laid on the table and forgotten. Whether it will be taken up this year no one knows, but the chances are that it will not be adopted.

The reason why a state-wide code is best is due to the rivalry between towns in attempting to make of each place a manufacturing center. Inducements are offered to manufacturers to locate, and they are promised immunity from taxes for a certain length of time, with gifts of buildings and sites and certain concessions in the way of payment of bills for water and light. The additional number of inhabitants secured, it is believed, will take care of these items. When the local chamber of commerce or board of trade offers a building to an incoming manufacturer, there is a desire to put up this building at as low a cost as possible, and put up the very poorest building the manufacturer will consent to occupy. A town offering inducements to manufacturers may have a very complete code, equal in all respects to the large building code approved by the National Board of Fire Underwriters. If a prospect arises for securing a manufacturing establishment, the employees of which will nearly double the population of the town, it requires only a special meeting of the council and a very short resolution to set aside the provisions of the building code in so far as it relates to that particular building. The worst feature is the fact that the entire community applauds such action. So long as such a condition can exist, the subject of building codes in small towns should be left in the hands of the local architects and builders and local representatives of the larger insurance companies.

TREATMENT OF CONCRETE ORNAMENTAL ELEVATED STATIONS, DUAL SYSTEM OF RAPID TRANSIT, NEW YORK CITY.

By S. J. Vickers.*

The City of New York, by the Public Service Commission of the First District, is now constructing what is known as the Dual System of Rapid Transit.

The new work ties in with the present Rapid Transit System giving over 600 miles of single track with a carrying capacity of 3,000,000,000 passengers per annum and will cost $350,000,000.

Four of the five boroughs are linked together with tunnels and all four of the bridges are utilized by the system. Manhattan has no elevated work

FIG. 1.—PELHAM PARKWAY STATION ON THE NEW YORK RAPID TRANSIT LINES.

belonging to this system, but a number of the subway lines extending into the outlying boroughs become elevated lines.

It is the policy of the commission to beautify stations which we are constructing near parks or at the intersection of important streets forming points of interest or civic centers. These stations for the most part are steel structures, encased with concrete. It is the treatment of these concrete structures which forms the subject of this paper.

Our first specifications called for the concrete to be filled in with special mix on all outer or exposed surfaces. This was to be kept from the ordinary concrete by steel shields placed about 2 in. from the outer faces and moved

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up as the work progressed. This was called for because it was believed we would be unable to get the color and texture desired in the ordinary concrete.

This method was, however, never used, as it was found that by using Long Island gravel, which for the most part is light in color but contains some dark stones which gives a variety, and by properly dressing the surface, excellent results could be obtained. We therefore promptly gave up the shields and special mix, thereby avoiding considerable structural difficulty.

In pouring the concrete, care was taken in getting a uniform mix and working it tightly down against all faces of the forms. Otherwise on their removal there would have been ill-formed corners and honeycombed surfaces. There is little hope of filling up these imperfections thereafter so that they will not show when dressed. Even when extraordinary care is used, certain imperfections will appear after the removal of the forms. In certain places it was necessary to cut these imperfections out to a considerable depth and fill with a concrete the same as the original mix, which was 1 : 2 : 4.

The inside of the forms were given a coat of cheap lubricating oil pre-

FIG. 2.—MOSHOLU PARKWAY STATION ON THE NEW YORK RAPID TRANSIT LINES.

pared especially for this purpose. In certain work we were able to have the forms removed within a week.

No dressing was done to the concrete until after it had been set at least a month, as it was desired in the surfacing to chip the gravel to give the surface a bright and sparkling appearance.

On all of our stations the dressing was done with pneumatic machines. The four-pointed tool was used on the greater part of the surfaces, but in certain large fields where a rough surface was desired the bull-point or one-pointed tool was used. Bands varying from 2 in. to 4 in. were usually left around all panels and corners which were rubbed smooth, giving a frame or setting to the regular work. On certain work where there were big surfaces, a wider band was left and this was hand-tooled with lines at right angles about $\frac{3}{4}$ in. apart. These are practical ways of finishing corners or arrises, as it is impossible to dress them with a pneumatic tool without chipping and forming more or less irregular lines.

The contractors were required to dress away any unevenness caused

by the bulging of the forms, bringing the surfaces to even planes in which all the cement skin was removed and the gravel lying near the surfaces chipped or cracked to give a sparkling effect. By the removal of a considerable portion of the surface most of the board marks disappeared. It is a curious fact, however, that a few board marks will remain although the entire surface is dressed to a plane, caused probably by the fine sediment collecting about the joints of the boards.

The most unusual piece of construction with which we are concerned is the reinforced-concrete structure for Queens Boulevard in the Borough of Queens, which line extends along the wide boulevard for a distance of $\frac{4}{5}$ mi.,

FIG. 3.—NEAR VIEW OF ORNAMENTAL CONCRETE ON ELEVATED STATION.

and is so arranged that trolleys will run underneath the structure, either side of which are planned a driveway and a strip of park. From the large reinforced columns 65-ft. longitudinal arches and 35-ft. cross arches spring. The form of the vault supported by these arches is in appearance a dome. The entire surface including the domes of this structure was tooled except certain bands and arrises and margins of panels. A continuous band of colored tile with a plaque on each column was inserted to add a little color to the gray concrete surface.

About 20,000 sq. ft. of tile was used on this piece of construction. This is the largest tile contract of its kind of which I have any knowledge. The tile was selected with great care after considerable investigation. It is wet

pressed, hand-made, semi-vitreous, and glazed, with scores or dovetails on the back to insure a perfect bond with the concrete. The recesses left in the concrete for the insertion of the tile were about $1\frac{3}{4}$ in. deep and the tile was laid in $\frac{1}{2}$-in. gray joints.

In this connection I wish to call attention to the appropriateness of inlaid colored tile as an enrichment for concrete structures. It is not unusual to see concrete structures built in imitation of stone with cornices, modillions, dentals, string courses, rustications, keystones, all of which may only be done with expensive form work. At least one writer has gone to the trouble

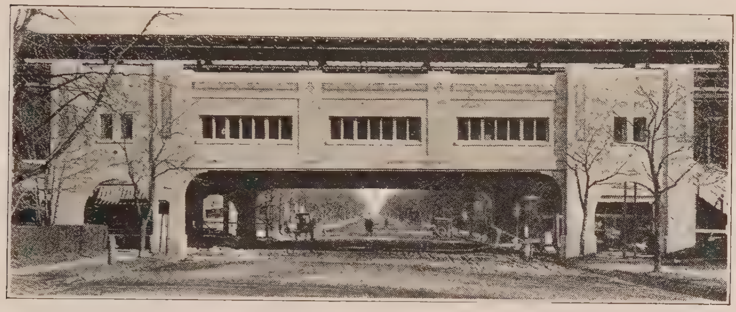

FIG. 4.—CONCRETE COVER OVER STEEL FRAME FOR OVERHEAD STATION.

to study these forms and see how they may be modified and used with less difficulty in concrete, apparently not realizing that these forms lose their significance when used in a plastic material. How much better to design simply, striving for big surfaces which may be enriched but unbroken by well-placed and well-designed inlaid tile.

In the designing of concrete we should omit meaningless forms, however appropriate they may be in other materials, striving rather for large and simple surfaces dressed to a uniform texture, enriched, if you like, but unbroken.

HISTORY AND PRESENT STATUS OF THE CONCRETE PILE INDUSTRY.

By Charles R. Gow.*

Prior to the beginning of the present century, piling for foundation purposes as used in this country was almost invariably of wood. It is true, of course, that special conditions had at times imposed the necessity of introducing substitutes for wood piling, and the use of iron and screw piles was more or less familiar to engineers of that period. Sand piles had been used in a limited way, and the writer recalls having seen light steel shells filled with concrete and buried in the ground as foundation supports during the late years of the last century.

It was generally recognized, however, that wooden piling although relatively cheap was poorly suited for many of its requirements. The necessity of cutting wooden piles below permanent ground water level frequently required excessive amounts of excavation and correspondingly large masses of masonry to bring the footings to grade. If the groundwater level was in any way liable to future depression, there was justification for more or less apprehension as to the future integrity of the structure which the pile supported. Again, the amount of loading which could be applied safely on wooden piles was comparatively small. The liability of breakage during the process of driving was also recognized by engineers, as was the fact that such breakage could not always be readily detected by means of surface indications.

These and other considerations have called for the exercise of ingenuity on the engineer's part in providing a satisfactory substitute for wooden piles which would eliminate their disadvantageous features. The general adoption of concrete as a building material during the late years of the past century naturally led to its introduction for various structural purposes and it was soon applied as a means to take the place of wooden piling. From that time on, the application of concrete to the piling industry enjoyed a rapid and encouraging growth, and today its use throughout the world has assumed very substantial proportions. The experimental stage has long since passed, and we now universally accept the concrete pile in its many forms as an available expedient for the solution of our physical and economic foundation problems.

In the development of the concrete pile industry there have ensued two distinct types or groups, viz., those which are pre-cast on the surface and later driven in substantially the same manner as are wooden piles, and the so-called "built-in-place" piles, which are cast in their final positions after the necessary opening in the soil has been obtained by means of temporary or permanent forms or shells. These two types of piles will be treated separately.

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PRE-CAST CONCRETE PILES.

It was natural that the earlier adaptations of concrete to piling should follow in form and methods the previous experience with wooden piles. Accordingly, concrete piles were first cast in molds of approximately the dimensions of heavy wooden piles, or somewhat larger, and in general a rectangular cross-section was adopted for convenience rather than the circular shape. The first piles of this type to be used, so far as there is record, were introduced in France by Hennebique during the year 1896, and their introduction into the United States followed some years later. Generally speaking, pre-cast piles are built with a square, hexagonal or octagonal cross-section. When the square section is adopted, it is customary to chamfer the corners slightly, in order to avoid the danger of chipping during handling. The piles are usually cast with a diameter of from 12 in. to 24 in., depending upon their length and the load they are to carry.

This type of pile is almost invariably reinforced with some suitable arrangement of longitudinal bars and circumferential hooping. In many instances the hooping consists of a spiral winding either of wire or small-sized round or flat bars. In a number of instances, sheets of electrically welded wire fabric have been utilized as reinforcement in piles of moderate length, the heavier wires of the mesh extending in the longitudinal direction, while the lighter strands serve as transverse hooping. In some instances, additional reinforcement is provided near the top of the pile to resist the tendency for the head to rupture under the impact of the hammer. Similarly, in the case of very long piles, extra longitudinal rods are used to strengthen the middle section against stresses due to preliminary handling. While the method described above is the customary practice in reinforcing these piles, other types of reinforcement may be and have been adopted, as, for instance, angle irons connected by lattice bars or even I-beams incased in concrete, the object being always to insure the necessary strength to withstand the stresses which may be induced in the pile due to handling, driving or loading. In the ordinary case, it usually will be found that the greatest stresses will be developed when the pile is picked up in a horizontal position and supported by means of a single fastening at the middle of its length.

Protection at the point is often afforded by the use of a cast-iron steel point or shoe (Fig. 1), which usually is cast integral with the pile and secured to it by means of adequate anchorage. The function of this special point is, of course, to prevent breakage at the tip of the pile during driving, which might result from encountering boulders or other obstructions.

In many cases a jet pipe is provided for in casting the pile, and may be readily formed by building in a length of ordinary wrought-iron pipe longitudinally in the center of the pile. This pipe may project sufficiently at the top to permit of attaching the discharge hose from a jet pump, and the lower end is usually reduced where it emerges from the point so as to form a nozzle. The writer has found that a cheap variety of tin speaking-tube will answer all purposes of the longitudinal passage in the center of the pile, and this tube may then be connected at either end by short pieces of wrought-iron pipe, which serve as hose connection and nozzle respectively. It is often found

convenient to introduce a water jet through a horizontal nipple in the side of the pile, a short distance below the head, and to connect this nipple with the longitudinal passage by means of a right-angle elbow. This arrangement eliminates certain difficulties in operating the jet during the driving process.

Pre-cast piles are sometimes built with a longitudinal taper. Sometimes they are of uniform section throughout, while still others are uniform in section except for a short length near the tip, which is tapered to give a long, wedge-shaped driving point.

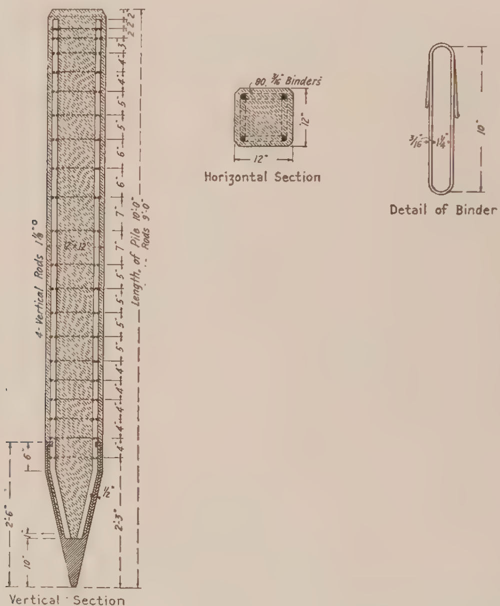

FIG. 1.—SQUARE (UNPATENTED) REINFORCED-CONCRETE PILE, USING STEEL PROTECTING POINT.

In a few instances, piles have been cast with a hollow section when it was desired to obtain a maximum of stiffness with a minimum of weight. These are naturally more expensive to build, and the type is not especially common.

None of the above features is patented, and any of them may be adopted at will. Such patents as have applied to the pre-cast type of pile are usually confined to one or more special features either in the details of design or in the method of making and driving. These features are supposed to afford better results in many respects than ordinarily could be obtained by means of the unpatented types.

Among the special patented types of pre-cast piles in more or less common use at the present time, in this country, may be mentioned the following:

The Chenoweth Pile.—The "Chenoweth" pile, patented in 1904 by Mr. A. C. Chenoweth, of Brooklyn, N. Y., depends chiefly for its patentable features upon the novel method of its manufacture. This and the resulting arrangement of its reinforcing metal give to it special properties which materially assist in facilitating its safe handling and driving. The pile is circular in cross-section, with a variable diameter from 12 in. upward, dependent upon the length of the pile required. This pile is formed in the following manner (Fig. 2):

A slightly inclined table is constructed of a length somewhat in excess of that of the pile to be made. This table is free to move laterally upon suitable rolls. Near the upper edge of the inclined table is secured a length of 2-in. pipe, capable of being revolved. To this pipe is attached one longitu-

FIG. 2.—CONSTRUCTION OF CHENOWETH PILE.

dinal edge of a sheet of expanded metal or fine wire mesh, which is spread flat upon the surface of the inclined table. Upon this sheet of metal fabric, longitudinal reinforcing rods are laid at intervals and wired to the mesh. A layer of moderately stiff concrete is then spread over the prepared reinforcement and shaped by means of a template so as to conform to certain necessary requirements. The 2-in. pipe is now revolved, drawing the table with its contents toward it as the combined layers of steel and concrete are wrapped about it. Simultaneously, a heavy roll immediately above the 2-in. pipe bears against the outer surface of the pile, imparting to it the required cross-section. After the rotation is completed, the pile is bound at frequent intervals with small wires. It will be seen that the resulting pile section resembles somewhat the arrangement of the layers in a jelly roll, the metal

fabric with its attached longitudinal rods forming the spiral hooping which affords great strength against stresses due to handling and driving.

The Corrugated Pile.—The "corrugated" pile (Fig. 3), patented in 1908 by Frank B. Gilbreth, of New York, is octagonal in section, with a uniform taper from 20 in. to 11 in. It is reinforced with a circular cage of Clinton electrically welded wire fabric. Its patentable features consist chiefly of longitudinal semicircular corrugations in the middle of each of its eight exterior faces and of a tapered cored passage in the center of the pile, which is formed by casting a wooden mandrel in the center of the pile having a 4-in. diameter at one end and a 2-in. diameter at the opposite end. The large end of this wooden mandrel projects sufficiently through the end of the form so that it may be given a slight turn after the concrete is partially set, thus breaking its bond with the concrete and permitting its easy withdrawal when the concrete is thoroughly hardened. The purpose of the corrugations is intended to be twofold. First, to afford a ready passage for the water returning from the jet pipe to the surface, and, second, to increase the total perimeter of the cross-section, thereby making possible a greater frictional contact with the soil. The cored passage in the center of the pile serves as a means for the introduction of the jet pipe and thereby eliminates loss which would ensue through building a separate pipe in the center of each pile. So far as the efficacy of the corrugations is concerned, it may be questioned whether they add in any way to the ease of driving. It would seem that the increased perimeter might better be obtained by the adoption of a square rather than an octagonal section, though it may be argued that an equivalent superficial area is obtained in this manner with a less amount of concrete. This type of pile enjoyed a certain degree of popularity for a time but has been little used of late years, so far as the writer is aware.

FIG. 3.—CORRUGATED CONCRETE PILE.

The Cummings Pile.—The "Cummings" pile (Fig. 4), patented by Mr. R. A. Cummings, of Pittsburgh, Pa., is distinguished from the unpatented types chiefly by the introduction of a special design of reinforcement in the head of the pile which avoids danger of breakage at the point. A series of circumferential and spiral flat bands is inserted in the head of the pile, and it was found by testing one of these piles to destruction, under repeated blows of a heavy hammer, that the pile ultimately failed by the breaking of the hooping in the body of the pile while the specially reinforced head remained intact, indicating that it is possible by some such means to secure sufficient strength to meet the severe stresses due to impact. On the other hand, with the adoption of suitable cushion caps, it has been found possible in the majority of instances to avoid the kind of breakage which this patented type provides against. Nevertheless, it is undoubtedly true that there are cases where very severe driving becomes necessary, in which case some such provision may be necessary if damage to the pile head is to be entirely avoided.

This type of pile was successfully used in the construction of the Farmville Bridge across the Appomattox Valley, on the Norfolk & Western Railroad where 60-ft. piles were driven through an old fill allowing only $\frac{3}{4}$ in. average penetration per blow. Under such conditions it is obvious that some special protection is needed to maintain the integrity of the pile head.

The Bignell Pile.—The "Bignell" pile (Fig. 5) is patented and is controlled by the Bignell Piling Company, of Lincoln, Neb. Its main patentable feature consists of a double jetting system. A 4-in. jet pipe extends longitudinally through the center of the pile, being reduced at the pile point to $1\frac{1}{8}$ in. This 4-in. pipe is tapped at frequent intervals and connected by means of $\frac{3}{4}$-in. nipples with the four outer faces of the pile. The outer extremities of these nipples are turned upwards by means of right-angle elbows. In addition, a 2-in. jet pipe is inserted inside the 4-in. pipe and connected with the $1\frac{1}{8}$-in. nozzle at the point. By this means two separate jet processes are carried on simultaneously. The 2-in. pipe operating under high pressure

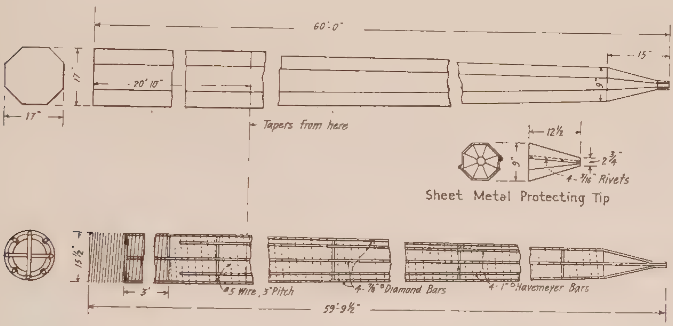

FIG. 4.—DETAILS OF CUMMINGS (PATENTED) PRE-CAST PILE.

delivers a jet stream in the usual way at the point of the pile, while the various $\frac{3}{4}$-in. connections permit water at a somewhat less pressure to escape upwards along the four faces of the pile, thereby destroying all frictional resistance and permitting the pile to sink into place by its own weight. This type of pile is usually handled by a derrick or crane, and no pile driver or hammer is deemed necessary. On the reconstruction of the Platte River Bridge of the Chicago, Burlington & Quincy Railroad, at Ashland, Neb., piles of this type were used. They were 15 in. square in section, 50 ft. long, and weighed 6.2 tons. Two independent pumps were operated, one to give a nozzle pressure of from 200 to 300 lb. per sq. in. and the second to deliver water to the side jets at from 100 to 150 lb. per sq. in. It is reported that the average time of sinking a 50-ft. pile in this manner was 42 minutes. The piles are reported to have cost 53 cts. per lin. ft. to manufacture, and 15 cts. per lin. ft. to sink. The total estimated cost of the completed pile was $1.00 per lin. ft. for piles not to exceed 50 ft. in length. It seems probable that this method of sinking might be quite satisfactory in certain classes of material, such as sand and

gravel. There might, however, be some question of the sufficiency of the jet method alone in many materials such as clay and hard pan or some classes of rough fill.

The Giant Pile.—The "Giant" pile (Fig. 6) is patented and controlled by the Giant Concrete Piling Company, of New York. Its patentable feature lies principally in the method of driving the pile. A cast-steel point is built integral with the pile. The upper flanges of this point project about 1½ in. beyond the concrete faces of the pile itself. A driving frame, consisting of two channel irons incasing two sides of the concrete pile for their entire length, rests at the bottom upon the projecting edges of the cast-steel shoe. A specially designed head is attached to the channel iron frames and receives

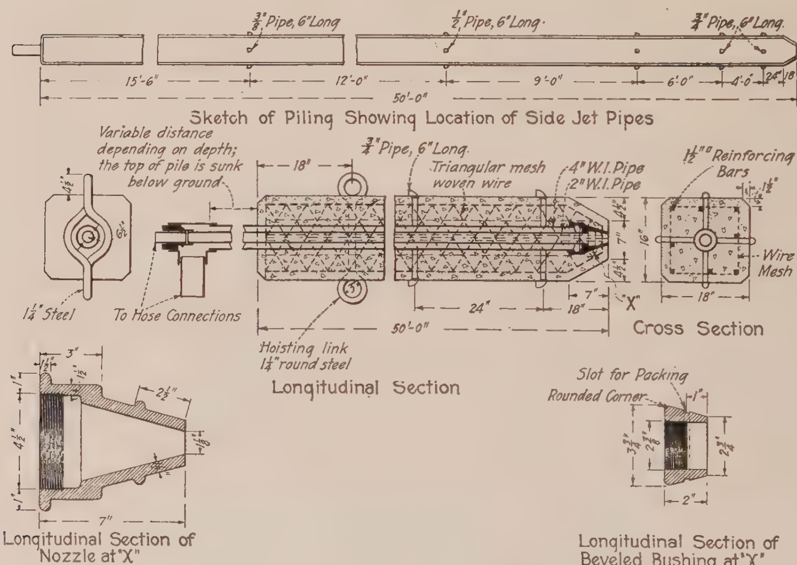

FIG. 5.—THE BIGNELL (PATENTED) PILE.

the blows of the hammer. By this means the entire force of the hammer blow during driving is transmitted directly to the cast-steel shoe, and no strains are induced in the concrete itself. When the piles are driven, the channel iron frames are withdrawn, to be used again on the next pile. The space which is left in the ground when the frames are withdrawn must be filled by the closing in of the soil against the surface of the concrete pile. Undoubtedly this arrangement permits of very severe driving without endangering the integrity of the pile itself. While it is probable that in a majority of cases the surrounding soil will close around the pile after the frames are withdrawn, it may be considered problematical whether the apparent friction observed in driving is reproduced after the frames are withdrawn.

METHOD OF MANUFACTURE OF PRE-CAST PILES.

Pre-cast piles are usually concreted in a horizontal position. In some of the earlier instances it was deemed necessary to cast them in vertical forms in order that the stratification of the concrete might be normal to the direction of the load. This method of manufacture, however, adds very considerably to the cost of the pile, and it is very doubtful in the writer's opinion whether there is any compensating advantage. It is the almost invariable custom in pile making to use a wet mixture of concrete which has very little if any stratification. It seems likely that, in cases where additional strength is required over that obtained on the ordinary horizontal method of casting, it might be more economically obtained by increasing the richness of the mixture rather than to adopt the very expensive method of casting in vertical forms.

Because of the ease and simplicity of form construction, the square section of pile has become, generally speaking, the most popular (Fig. 7), although hexagonal and octagonal shapes are still used (Fig. 8), but only to a limited extent. The square section also commends itself because of the greater surface area exposed to friction. If for any reason a circular section is desired, it can sometimes be secured by selecting a casting yard having a pronounced inclined slope so that the angle of the forms will permit pouring through the end, otherwise vertical forms will be necessary.

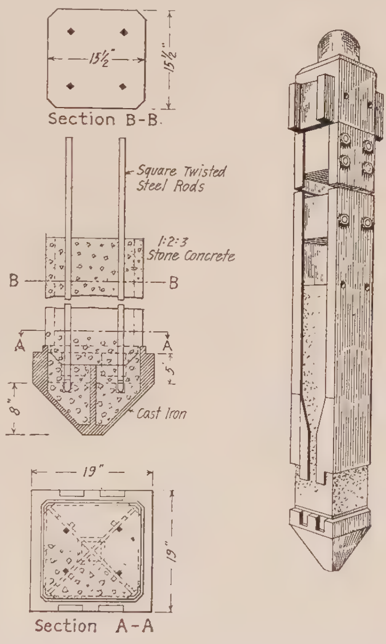

FIG. 6.—THE GIANT (PATENTED) PILE.

Because of the requirements of a homogeneous mixture of concrete and early strength, pre-cast piles are seldom or never made with a mixture leaner than 1 : 2 : 4, and occasionally a richer mix is deemed advisable. It is safe to say, however, that by far the great majority of pre-cast piles are now made of the first-named mixture.

It will be found desirable, in any given case, to devote a considerable amount of thought to the design and construction of the form work used in casting concrete piles. Above all, it is essential that an unyielding base should be provided under the forms, so that unequal settlements will not cause damage to partially set piles. Various and ingenious contrivances and arrangements of form work have been designed from time to time. Figs. 9 and 10 illustrate typical cases.

The fabrication of the reinforcement should also have careful attention. A simple contrivance for economically preparing spiral hooping is shown in Fig. 11. Extreme care is absolutely essential in placing the reinforcement. The reinforcing metal is usually assembled as a unit so that it may be lowered into the form and accurately centered before the placing of the concrete

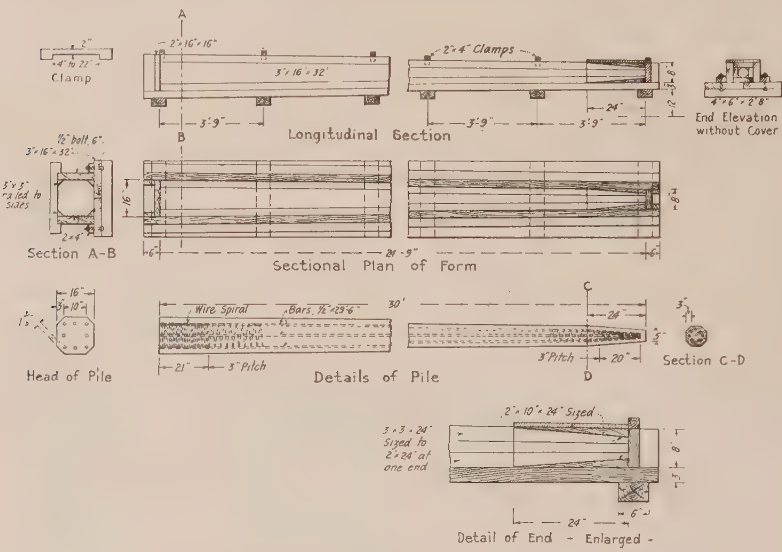

FIG. 7.—PRE-CAST CONCRETE PILE (SQUARE) AND FORM.

begins. If a metal point is to be used, it is fitted in place in the end of the form prior to concreting. The sides of the forms can generally be removed in from 24 to 48 hours after casting, and are then available for use in subsequent forms. Under normal weather conditions, pre-cast piles do not

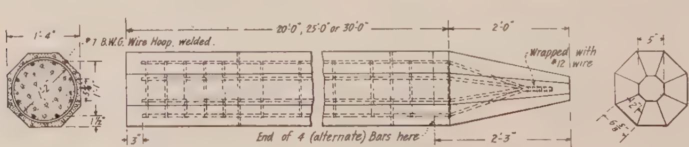

FIG. 8.—PRE-CAST OCTAGONAL PILE.

acquire sufficient strength to permit of handling or driving inside of 28 to 30 days after casting. This feature necessitates the use of a very considerable area as a casting yard in order to maintain a sufficient number of properly cured piles for driving at all times. It has been found that the proper aging of pre-cast piles may be materially accelerated by the adoption of steam curing methods. The piles are assembled in tiers and enclosed with some suitable

form of covering under which live steam is fed continuously. The combination of heat and moisture hastens the set of the concrete without injury, so that they may be handled and driven in some cases within a week of the time of casting.

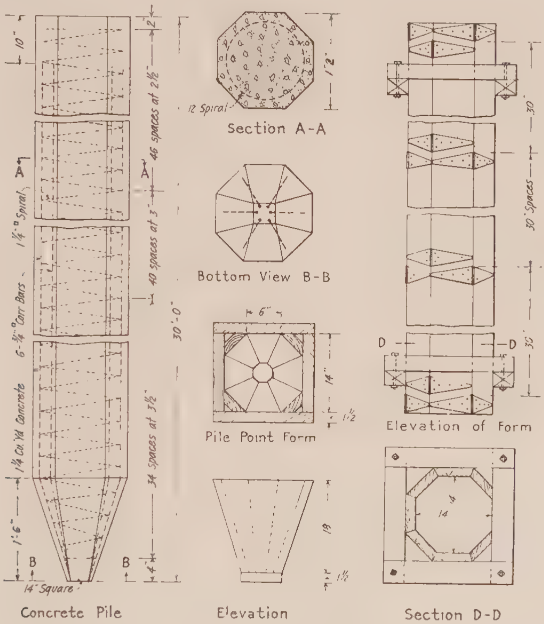

FIG. 9.—FORM FOR CASTING CONCRETE PILE.

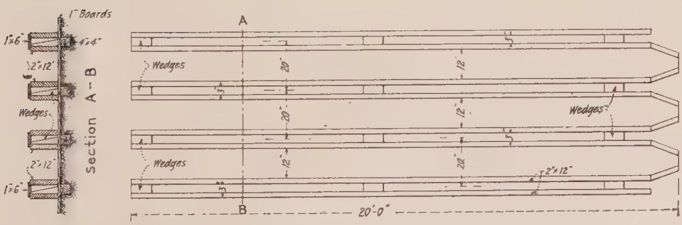

FIG. 10.—ARRANGEMENT OF FORMS FOR CASTING CONCRETE PILES.

Unless piles are adequately reinforced to resist rough handling in the preliminary stages of the work, a considerable degree of care is necessary in moving them to the driver and in raising them to a vertical position (Fig. 12). While piles in lengths up to 25 or 30 ft. may be designed to stand readily a maximum stress due to handling, it will be seen that the longer lengths would

require cross-sections of prohibitive dimensions if this provision were to hold in their case. It is, therefore, usually necessary in the handling of long piles to use special precautions which will relieve the pile from undue strain. This is generally possible by means of multiple supports during the lifting and righting operations. In the case of piles measuring from 30 to 50 ft. in length, it will be found possible to handle the pile successfully by means of a bridle chain attached at one point near the head and the other near the center of the pile (Fig. 13). In lifting, the strain is thereby distributed at the two points, and as the pile is raised into a vertical position no excessive force at any time operates to overstrain the steel or concrete. Piles in excess of 100 ft. in length and measuring 20 by 20 in. in cross-section have been successfully handled by similar methods except that two bridle chains were used, per-

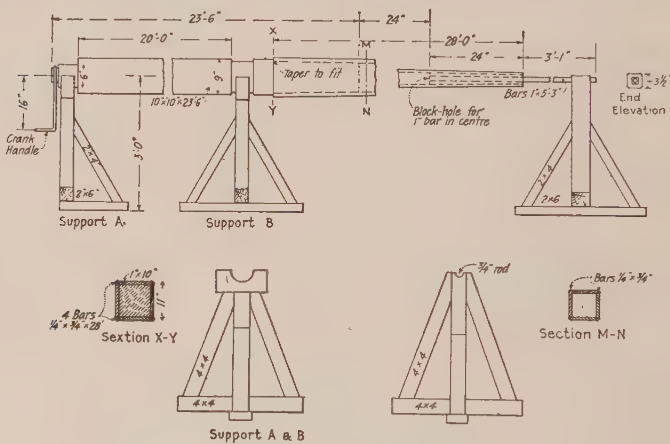

FIG. 11.—MANDREL FOR WINDING WIRE COIL FOR REINFORCING CONCRETE PILES.

mitting support at four points on the pile (Fig. 14). The rule which seems to be in most common use for lifting long piles has been to attach the hoisting falls at points 20 per cent of the pile length from either end (Fig. 15). In some cases it has been found convenient to insert pipe nipples transversely through the pile section at the points where the lifting strain is intended to be applied (Fig. 16). A pin or bar may then be inserted in the hole thus formed, which assures the application of the lifting strain at the desired point.

One of the drawbacks in the way of using concrete for the purpose of piling is the inability of the material to withstand heavy direct blows without fracture. This difficulty, however, has been largely overcome by the introduction of certain protecting devices which either resist the breaking stresses or cushion the blows to such an extent that the hammer impact has no harmful effect. Naturally much energy is lost by the introduction of this cushion between the hammer and pile head, therefore the least cushion required, the

more effective the hammer blows become in causing pentration. For this reason many designers of pre-cast piles introduce additional reinforcement in the head of the pile. In the construction of the St. Louis Viaduct, steel bands 12 by 12 by $\frac{1}{8}$ in. were cast integral with the head of each pile. Although there had been much breakage of pile heads on this job prior to the adoption of this banding method, no breakage occurred in the case of subsequent piles. At the normal price of steel plate, this form of protection would add not more than four cents per linear foot in the case of 25-ft. piles. The "Cummings" pile, as previously noted, is especially reinforced in a most effective manner against breakage at the head. Even in these cases, however, if hard driving is required, some form of cushion will nevertheless be required if damage to

FIG. 12.—HANDLING 51-FT. PILE WITH CENTER SUPPORT ONLY.

the pile is to be entirely avoided. Various types of driving caps have been devised for this purpose, and the essential features of all such caps are similar. They are made of steel or cast iron, and contain a recess in the bottom which fits the head of the pile, and a depression in the top in which is usually inserted a hard wood block to receive the blows of the hammer. Between the wooden block and the head of the pile is placed a layer of some cushioning material such as sawdust, sand, rope mats, or layers of rubber hose or belting. In general, the cast-iron caps seem to have given the most satisfactory service, and where the cap has been made of riveted steel plates there has seemed to be more or less difficulty due to the loosening and shearing of the rivets. The wooden blocks require frequent replacement, but otherwise the cap is

FIG. 13.—RAISING PILE TO VERTICAL POSITION.

FIG. 14.—BRIDLE CHAINS FOR HANDLING LONG PRE-CAST CONCRETE PILES.

used continuously. It is customary to equip these caps with a wire-rope strap, by means of which they may be temporarily attached to the hammer and thus raised or lowered from the pile head. It is also desirable to construct

FIG. 15.—BRIDLE CHAINS FOR HANDLING CONCRETE PILES.

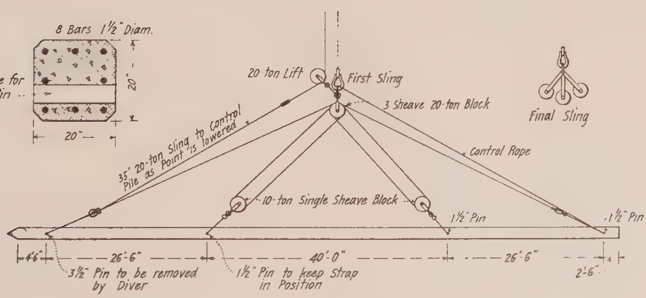

FIG. 16.—METHOD OF SLINGING 100-FT. CONCRETE PILES USED AT AUCKLAND, NEW ZEALAND.

the cap with some form of grooves which fit the leads of the pile driver and thus cause it to guide the head of the pile. Details of construction of various types of driving caps are shown in Figs. 17, 18 and 19.

TYPES OF HAMMER USED.

There has been much controversy as to the type of hammer best adapted for driving concrete piles. Some authorities have maintained that the drop hammer is superior for the purpose to the steam hammer. Other experienced pile drivers maintain that the steam hammer excels, and among the steam hammer adherents there is some disagreement as to whether the single or the double acting hammer is to be preferred. After a consideration of all the arguments pro and con, on this question, it seems reasonably safe to assume that, whichever type of hammer is used, the heavier it is the more successfully it will drive the pile. Because of the great mass of the pile itself, it is obvious that a large proportion of the force of the hammer blow will be expended in

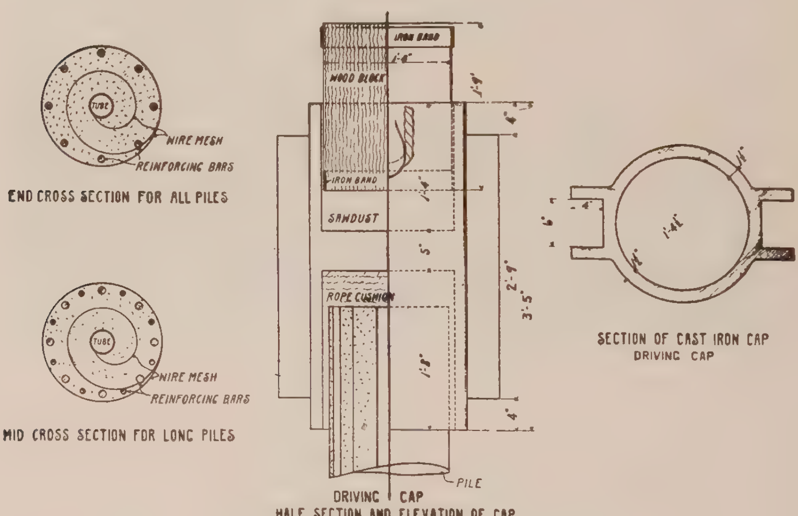

FIG. 17.—TYPE OF DRIVING CAP.

overcoming its inertia; therefore light hammers of either variety should be avoided in the driving of concrete piles. It seems to the writer a significant fact that, in the very great majority of concrete pile-driving jobs, the steam hammer is selected in preference to the drop hammer, and it would appear to be a fair presumption that it must on the whole, therefore, possess advantages over the drop hammer.

As to the double-acting steam hammer, it has not apparently enjoyed any considerable degree of popularity in the driving of concrete piles up to this time. It is reported that, in the driving of concrete piles for the St. Louis Viaduct, a 5000-lb. double-acting steam hammer was used at the beginning, and much difficulty was experienced because of the frequent breaking of pile heads. Finally the ram itself was broken whereupon a No. 1 Vulcan 10,000-lb.

single-acting hammer, with a 5000-lb. ram and a $3\frac{1}{2}$-ft. stroke, was substituted, and no further difficulty from breakage was experienced. The progress with the double-acting hammer had averaged 9 piles per day, and this output was increased to 17 per day after the introduction of the heavier single-acting hammer. With regard to the breakage of pile heads referred to, it is not clear why the type of steam hammer should cause such a difference, and it seems possible that the subsequent immunity from breakage might more properly be attributed to the use of the steel bands previously referred to in connection with these piles. The economy of the heavy over the light hammer, however, is well illustrated by the increased progress shown in this instance. The

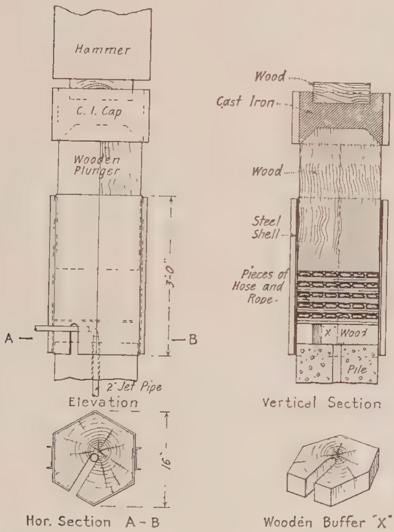

FIG. 18.—TYPE OF DRIVING CAP.

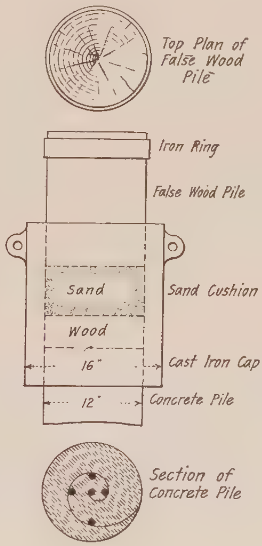

FIG. 19.—TYPE OF DRIVING CAP.

heaviest hammer so far used for this purpose was employed in driving pre-cast piles for Pier No. 2 of the Intercolonial Railway, at Halifax, N. S. The piles were 24 in. square, 57 to 77 ft. long and driven to rock, the largest pile weighing 23 tons. A 28,000-lb. double-acting steam hammer was used in driving them. From 200 to 1800 blows were required to drive these piles to rock, final penetration being $\frac{1}{30}$ in. It is reported that only two pile heads were broken, out of the 1800 piles which were driven on this job. The heads of the piles were protected by 3-in. spruce planks, upon which rested a cast-steel follower of hollow pattern and with top and bottom flanges. The top flange contained a depression in which was placed a wooden block 15 in. thick and bound with steel bands. Some blocks lasted for the driving of only two piles, while others served as many as twenty. They usually failed by the

bursting of the bands. An average of 10 piles in 10 hours was maintained on this work, with a maximum rate of 18.

In all cases where a drop hammer is used, the best results may be expected by employing as heavy a hammer as possible and using a small drop.

Another question which appears to remain somewhat unsettled is that of the relative merits of the straight and tapered pile. Undoubtedly the tapered pile may be driven more easily than the straight pile, especially when the water jet is not employed, and it also seems reasonably clear that if the entire length of the pile rests in a homogeneous material, the tapered pile may give equally satisfactory results as a support for the load it carries. The fact is, however—no matter how much we may theorize—that the primary object in using piles ordinarily is to carry the load through an unsatisfactory type of soil and transmit it to some more reliable stratum at a lower depth. It is not, generally speaking, conservative practice to attach any value whatever to the frictional supporting power of the unsuitable strata immediately below the surface, and, as a matter of fact, we are chiefly dependent for our supporting power upon the amount of penetration which the pile has in the good material below. It seems, therefore, to be highly desirable that the pile shall have its maximum section and frictional surface area at this level where it will do the most good. The writer is, therefore, inclined to agree with the adherents of the straight-sided pile. This, of course, does not preclude the tapering of piles at the point or even a short distance above the point, and in many classes of soil such pointing is desirable in order to facilitate easy driving. It cannot be denied, of course, that tapered piles have been successfully driven in innumerable cases with entirely satisfactory results, but in the writer's estimation this of itself does not justify an opinion that they are superior in supporting power to the straight pile, the displacement of which is greater, the soil compression consequently being greater, and the resulting surface friction between the ground and the pile being also, in his opinion, much in excess of that of the tapered pile.

It has become a more or less general practice, in the driving of concrete piles, to utilize the assistance afforded by the water jet as an aid to rapid and effective driving. This expedient is especially helpful when the material through which the pile is driven consists of sand or gravel, materials which offer a maximum of resistance in their normal state and which present little resistance when eroded by the force of a powerful water jet. The efficiency of this jet in driving through clayey soils is not so marked, and much greater force is required to displace this material. Ordinarily, a pressure of from 100 to 150 lb. per sq. in. is ample for jetting piles. As previously indicated, it is sometimes customary to provide a water passage by means of a pipe or other longitudinal opening through the center of the pile. The delivery hose from the hydraulic pump may then be attached at the top of the pile to deliver the water through a nozzle at the point. Some authorities maintain that better results are obtained when two jet pipes are used on opposite sides of the pile. A committee of the American Railway Bridge and Building Association has recently compiled information with respect to the methods adopted by various railways in driving piles. The use of the water jet was found to be more or less general, and was used chiefly in sandy or gravelly soils or

where deep penetration is required. It was found that the jet pipes used varied in sizes from $2\frac{1}{2}$ to $1\frac{1}{4}$ in., and nozzles from no contraction to $\frac{1}{2}$ in. in diameter. The water pressure used varied from 100 to 225 lb. per sq. in., and pumps with 3 to 4-in. diameter discharge pipes were generally used. A large volume of water at moderate pressure was stated to give the best results. It was also reported that on the St. Paul, Minneapolis & Omaha Railway a steam jet was sometimes used in place of water. This method was found to work satisfactorily.

When a water jet is employed in connection with the driving of concrete piles, it will be found advantageous to churn the pile up and down in the

FIG. 20.—SINKING PILE BY WATER JET AND "CHURNING."

hole made by the jet. In such cases the hoisting fall of the pile driver is attached to the head of the pile, the jet pump started, and when the jet has begun to operate, the pile is lifted 2 or 3 ft. and allowed to drop of its own weight. In this manner the soil is scoured from below the pile and softened to such an extent that it permits the falling pile to penetrate a substantial distance at each drop. In sandy soils it is usually possible to sink a pile for the greater part of its length in this manner, and driving is then only required for the last few feet of penetration (Fig. 20).

In some instances, where the soil possesses some cohesion, it will be possible to make a vertical hole in the ground with the jet alone into which the pile may be dropped for the greater part of its length. This method will be

quite applicable when the soil is composed of fine sand with a slight amount of clay in it.

It is often desirable, in wharf or pier construction where piles are subjected to the force of the waves, to anchor them in some manner which will prevent their becoming loosened. With this idea in mind, bulb-shaped footings 3 ft. in diameter and attached to concrete piles have been successfully jetted into place through sand.

Sometimes it becomes necessary to drive concrete piles through very rough material containing boulders or other obstructions. In such cases a small-sized pilot pipe may be driven into the ground until it meets the obstruction. A charge of dynamite may then be inserted and exploded with the purpose of breaking up the rough ground and permitting the pile to be driven through it.

BEARING POWER OF PRE-CAST PILES.

The supporting power of a pre-cast pile will depend upon a variety of factors. If the piles are driven to ledge, they become columns, so far as their load capacity is concerned, except that in general they are supported against lateral deflection and also they undoubtedly transmit at least a portion of their load by means of friction to the soil above the ledge. In such cases the load should be apportioned to the piles in proportion to the safe compressive value of their minimum sectional area. If the amount of penetration of the pile in good material is small, it will be seen that a tapered pile figured as a column will suffer in comparison with the straight pile so far as its safe supporting value is concerned. It is sometimes customary to figure the aggregate capacity of a column pile as equal to the combined safe compressive values of both the steel and concrete in its sectional area. Figured on this basis it will be apparent that a pile of relatively small sectional area may be permitted to carry a very considerable load as a column, secured as it is against lateral deflection. It has already been stated that the piles for the Halifax Terminal Pier, which were 24 in. square in cross-section and up to 77 ft. in length, driven to rock, were loaded with 100 tons each. The same considerations which apply to pre-cast piles bearing upon ledge may with slight modifications be assumed to hold in cases where the pile is driven to an unyielding hardpan overlying ledge. In either of these cases it will be obvious that the straight pile is superior to the tapered pile in point of safe carrying capacity.

When the supporting power of the pile depends entirely upon its frictional contact with the surrounding soil, there is far less certainty as to either its ultimate or safe load capacity. The same theory is usually applied and the same formula generally followed as in the case of wooden piles. The so-called *Engineering News* Formula, $L=\frac{2\,Wh}{S+1}$, where L equals the safe load in pounds, W equals weight of hammer in pounds, h equals fall of hammer in feet, and S equals penetration in inches under the last blow, has been extensively employed to determine the safe load which the piles are capable of supporting. There is an essential difference, however, between the wood and concrete piles which doubtless requires some modification of this formula. The rela-

tively large mass of the pile itself consumes a considerable portion of the energy of the blow in overcoming its inertia. It has been suggested by some authorities that the *Engineering News* Formula, if used, should be modified so as to take into account this factor, and the particular formula which has been proposed is $L = \dfrac{2\,Wh}{S\left(1+\dfrac{w}{W}\right)}$, where w equals the weight of the pile and W that of the hammer. The formula for single-acting steam hammer which has been generally adopted, viz., $L = \dfrac{2Wh}{S+.1}$, should presumably be similarly modified, in the case of concrete piles, to $\dfrac{2\,Wh}{S\left(1+\dfrac{.1w}{W}\right)}$. All formulae which have so far been proposed in connection with the use of the double-acting hammer are more or less unsatisfactory because of the difficulty of accurately determining the mean average steam pressure during the downward stroke of the piston. It probably would be safe in any event not to rely too much upon formulae for load capacity, but to decide the question upon the results of observed tests and apply the results thus obtained to the formulae in order to secure a coefficient which may be made to apply to the driving of subsequent piles on that particular job or under similar conditions. In case actual loading tests are not made, it will probably be well to limit the allowable load on ordinary types of concrete piles to a maximum of thirty tons. This assumed value seems justified in view of the fact that the various available records of test loads upon concrete piles having a reasonably good penetration into satisfactory soil have failed to disclose any failures due to settlement under loads below this amount.

RATE OF PROGRESS IN DRIVING PRE-CAST PILES.

No rule can be laid down for determining in advance the probable progress to be expected in the driving of pre-cast concrete piles. Just as various attending conditions will vary the rate of progress in the driving of wooden piles, so also the number of concrete piles which may be driven in a given period will fluctuate accordingly. Undoubtedly the use of the water jet has a marked influence in increasing the rate of driving. So also does the adoption of the heavier types of hammer. Even with these facilities, soils are often encountered which refuse to yield appreciably under their influence and recourse must be had to long-continued driving with small increments of penetration. Generally speaking, however, under the most severe conditions of driving, and taking a 25-ft. pile as a fair average of length, not less than six piles may be estimated as a day's work, while, with favorable conditions, as many as thirty to forty such piles have been driven by a single machine in one day. It may be assumed, therefore, that the expected daily average will lie somewhere between these limits; and with a prior knowledge of local conditions it will usually be possible to classify the relative difficulties so as to determine approximately what the probable output is likely to be.

PRESENT GENERAL PRACTICE IN THE DESIGN OF PRE-CAST PILES.

The Masonry Committee of the American Railway Engineering Association reported, about a year ago, on the current practice of several railroads in their reinforced concrete pile construction. It stated, among usual customs, that the transverse reinforcement used was generally $\frac{1}{4}$-in. rods, arranged either as a spiral or spaced hooping. Steel wire mesh was used for reinforcement in a few cases, and additional steel was sometimes used in the pile head. The weight of the steel per linear foot used by the several railroads ranged from $4\frac{1}{4}$ lb. on the Chicago, Burlington & Quincy Railroad, to $12\frac{1}{2}$ lb. on the Illinois Central and $17\frac{1}{2}$ lb. on the Chicago, Milwaukee & St. Paul Railroad. Usually,

FIG. 21.—HANDLING AND DRIVING CONCRETE PILES.

16 in. was the short diameter adopted, whether the section was square or octagonal. The Atchison, Topeka & Santa Fé Railroad used the formula $D=7''+\frac{1}{4}''\times L$, where L equaled the length in feet and D the diameter of the pile in inches. The use of straight piles was found to predominate over the tapering type.

COST OF PRE-CAST PILES.

In common with all classes of underground construction, the cost of reinforced concrete piles varies with design and conditions. The cost of materials for any given case may usually be determined with reasonable accuracy for any given locality. The labor cost of making and driving will vary widely, depending upon various contributory causes.

On the St. Louis Viaduct work, where the piles were $13\frac{1}{2}$ in. square and 25 ft. long, four $\frac{3}{4}$-in. round bars hooped with No. 8 spiral wire on 2-in. pitch were used as reinforcement. The labor making up the unit reinforcing cages cost from $1.00 to $1.25 per pile, with labor at 65 and 70 cts. per hour. The cost of labor for casting these piles was 8 cts. per lin. ft., with labor at 40 cts. per hour. On the construction of the Yolo Bypass Trestle in the Sacramento Valley, Cal., 14-in. piles 32 to 50 ft. long were used, and the average cost of assembling and placing reinforcement, consisting of four $\frac{7}{8}$-in. round rods with spiral hooping, was $7.50 per ton. An average of 20 piles per day were cast, and 600 sets of forms were required, there being a total of slightly under 3000 piles; 177,000 ft. board measure of lumber was used for forms exclusive of mud sills. The cost of loading and hauling piles on a 3-ft. narrow-gage track for an average distance of $2\frac{1}{2}$ miles was 7.3 cts. per ton mile *for labor only.*

The writer on one occasion kept a careful record of the cost of making and driving 412 concrete piles ranging in length from 8 to 40 ft. They were cast with a 13-in. square section reinforced by a cage of No. 3 Clinton Wire Cloth placed 2 in. from the outer surface. They were sunk with the assistance of a water jet introduced through a hole in the center of the pile formed by casting a line of $1\frac{1}{2}$-in. speaking tube in the longitudinal axis. The concrete mixture used was 1 : 2 : 4. The soil consisted of mud, fine sand and some floating masses of clay hardpan. The piles were cast under cover in a sectional shed which was moved from time to time as required. They were handled by guy derricks, were held in upright position by a light portable timber framework which contained a pair of hammer leads, and were driven with a 2800-lb. hammer operated by the derrick falls (Fig. 21). Twenty piles was the maximum day's work. A sounding was taken at the location of each pile, to determine accurately the necessary length of the pile in order that it should reach ledge. There was a total of 8937 lin. ft. of piles cast and driven. The cost of the work in detail was as follows:

	Per Lin. Ft.	
Labor making soundings	$0.02	
Labor on forms	.035	
Labor casting, including placing reinforcement	.129	
Labor driving	.142	
Labor, Superintendent and miscellaneous	.128	
		$0.454
Materials for forms	.036	
Clinton wire reinforcement, $0.05 per sq. ft.	.167	
$1\frac{1}{2}$-in. speaking tube for center jet hole	.026	
Concrete aggregates	.212	
Materials used for heating and covering	.025	
Ropes, blocks, chains, etc	.044	
Coal	.023	
Miscellaneous supplies	.042	
Total for materials and supplies		$0.575
Total cost for labor and materials		$1.029

As this work was prosecuted in the winter time, it was somewhat more expensive than it otherwise would have been. The price per hour for labor at the time this work was done averaged about 20 cts.

Under the most favorable conditions it is unlikely that any pre-cast pile can be made and driven today at a contract price below $1.00 per lin. ft., and the cost is likely to range from this minimum to a maximum of $1.50.

NOTABLE INSTANCES OF THE USE OF PRE-CAST CONCRETE PILES.

In addition to the case cited of exceptionally heavy concrete piles used at the Halifax Terminal Pier of the Intercolonial Railway, where piles upwards of 77 ft. in length and weighing 23 tons were successfully handled and driven, it may be noted that pre-cast piles of a maximum length of 106 ft. and 20 in.

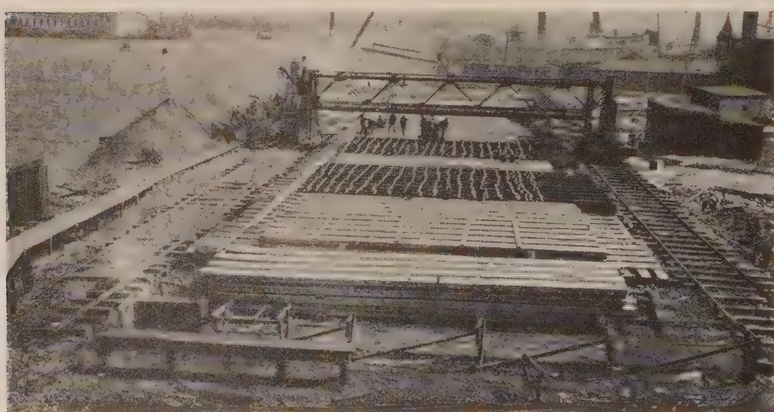

FIG. 22.—CASTING YARD FOR CONCRETE PILES.

square were driven in the construction of Pier No. 35 of the San Francisco Harbor Development. This extreme length was made necessary because of the very considerable depth of alluvial deposit. These piles were designed to carry 40 tons each. Pre-cast piles 85 ft. in length and 20 in. square were used in the construction of piers in Havana Harbor in 1914. They were driven by a No. 1 Vulcan steam hammer, with an average of 500 blows per pile, and brought up with a penetration of $\frac{1}{8}$ in. They also were designed to carry a load of 40 tons per pile. Pre-cast piles 100 ft. in length were used in the construction of wharves at Auckland, New Zealand. They were driven through 50 ft. of mud and clay to rock. These piles were 20 in. square, reinforced with 8 long rods and weighed a maximum of 20 tons.

So far as the writer is aware, there have been reported no failures on the part of pre-cast piles to fulfill their intended purpose as foundations. In his opinion, considered solely from the standpoint of efficiency, the pre-cast pile is superior to the other types. They possess, however, some serious dis-

FIG. 23.—CASTING YARD FOR CONCRETE PILES.
(Handling forms.)

advantages which render their use less common than it would otherwise be. With the modern tendency to hasten construction at every stage, the loss of time necessary to permit pre-cast piles to be properly made and cured renders their use prohibitive in many instances. Ordinarily it may be assumed that at least one month will be required after the work is begun and before actual driving can commence. As previously stated, this period may be very materially shortened by adopting the steam curing method when adequate means are available. A second serious drawback which limits the use of pre-cast piling, especially in congested districts, is the necessity of providing a very

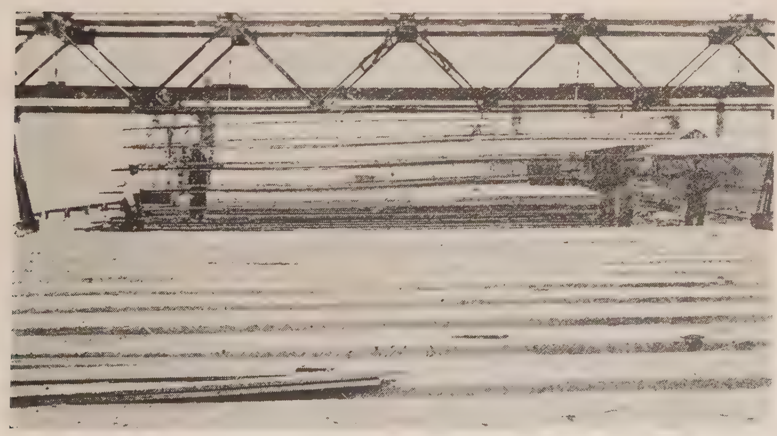

FIG. 24.—CASTING YARD FOR CONCRETE PILES.
(Handling reinforcements.)

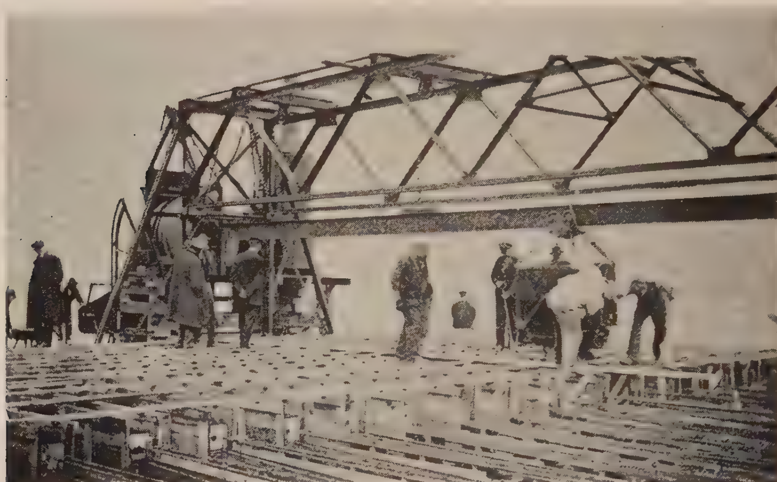

FIG. 25.—CASTING YARD FOR CONCRETE PILES.
(Placing concrete.)

considerable area for a casting yard. In the case of the ordinary city building work it is seldom possible to set aside a sufficient area for this purpose and, therefore, if pre-cast piles are to be used in such instances, they must be made at another point and transported to the building site as required. A very ingenious and efficient arrangement for reducing the space necessary for a casting yard is shown in Figs. 22 to 26. This system was devised and used

FIG. 26.—CASTING YARD FOR CONCRETE PILES.
(Handling piles.)

by Mr. W. L. Miller, of Boston, in connection with a concrete-pile contract for a pier at Baltimore, Md. It will be noticed that the casting yard is long in proportion to its width and that a traveling bridge or crane spans the space occupied by the piles. Its power equipment is located at one end of the trussed frame, while the concrete mixing plant is integral with the opposite end. By means of a long shaft, the crane is propelled longitudinally of the casting yard so that it may readily cover any section. This traveling gantry was used to handle forms, reinforcement and concrete, and, what is of still more importance, it was utilized to nest the partially cured piles in tiers so as to economize in space.

CAST-IN-PLACE PILES.

Because of the unsuitability of pre-cast piles for the conditions oftentimes met with, there have been introduced a variety of ingenious methods for constructing concrete piles in place. This method obviates the necessity of a large casting yard and of the preliminary delay necessitated because of the initial curing period.

The Raymond Pile.—The first pile of this type to be introduced in the United States was the "Raymond" pile, invented in 1896 by Mr. A. A. Raymond. This pile was first used in Chicago in 1901 (Fig. 27). It consists of a light, sheet metal tapered shell approximately 20 in. in diameter at the top and 6 in. at the bottom. This shell is fitted over a collapsible mandrel or core. The mandrel and shell are driven simultaneously by means of a heavily constructed pile-driving apparatus, and, when a satisfactory minimum penetration has been secured, the core is collapsed by an ingenious contrivance so that it may be withdrawn, leaving the shell to retain the surrounding earth while concrete is deposited within it.

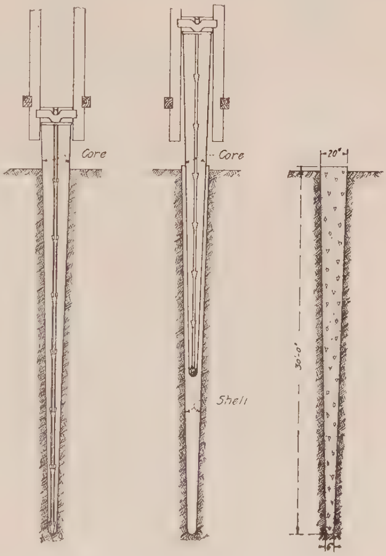

FIG. 27.—THE RAYMOND PILE.
(Built in place.)

The shell is usually built in 8-ft. lengths in such a manner that each succeeding section will lap the preceding one. The shells first used were of No. 18 or No. 20 gage, and it was frequently found that the reaction of the soil was sufficient to collapse this light metal wall after the core was withdrawn and before the concrete filling could be placed. In such cases it became necessary to drive additional shells inside the first until a combined strength sufficient to withhold the soil pressure was obtained. To obviate this difficulty, the

type of shell was altered, some years ago, in a manner which permitted the use of No. 24 gage metal corrugated spirally, while in the corrugations was placed $\frac{1}{4}$-in. steel wire which not only resisted the collapsing pressure of the surrounding soil, but in addition served as a spiral hooping around the concrete.

The cores first used were made in three longitudinal sections which were expanded by means of a central longitudinal rod with connecting links. As this device was somewhat delicate for the rough usage accorded it, a more substantial type of core was later devised, consisting of two heavy semicircular longitudinal sections with a flat center strip between them containing wedge-shaped wings which forced the two halves of the core apart or drew them together as might be desired.

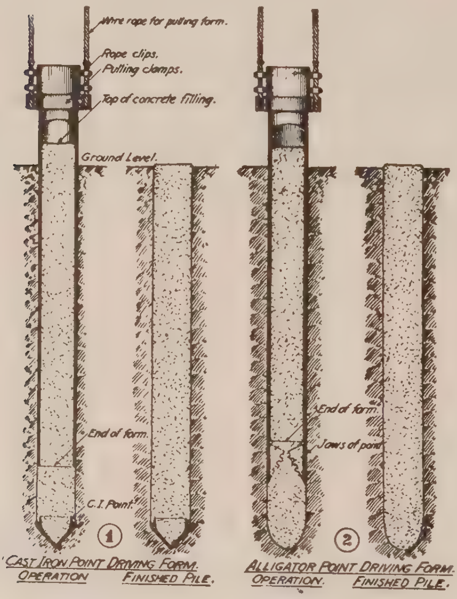

FIG. 28.—THE SIMPLEX PILE.
(Built in place.)

The advantages of this type of pile are found in the readiness with which its interior may be inspected before filling and the rapidity of progress due to the fact that the pile driver may proceed with the driving of a second pile while the first one is being filled with concrete. The concrete is protected by the spiral reinforcing against lateral strains during its period of setting. Reinforcing bars may be inserted before or during the concreting operation if desired, but this is seldom deemed necessary.

Among the objections urged against this type of pile are the following: Because of its pronounced taper, a section of comparatively small area penetrates the firm soil upon which chief reliance is placed for supporting the load. This is especially objectionable when the pile is driven to rock or other very hard strata, and in almost every case the larger proportion of the surface area of the pile will rest in unreliable top material. It is also urged that there is some danger, while withdrawing the core, of the shells being started upward with it. Because of their pronounced taper, such an occurrence even though small in amount would destroy the frictional contact between the shell and ground. This does not seem to the writer a very probable occurrence or necessarily serious one, unless by chance the bottom section should be so

withdrawn and thus remove the bearing at the tip. These piles have been driven up to 45 ft. in length.

The Simplex Pile.—In 1903 there was introduced in this country a built-in-place pile known as the "Simplex" pile (Fig. 28), the invention of Mr. Frank Shuman, of Philadelphia. It consisted at that time of a pre-cast concrete point resembling somewhat in shape a large projectile approximately 16 in. in diameter, upon which rested a steel cylindrical tube or pipe 16 in. in diameter and of sufficient length and strength to permit driving it to the required depth. The point and tube were assembled in a vertical position and driven by an ordinary pile driver to refusal. Powerful pulling tackle was then applied to the top end of the tube, and, as concrete was deposited in the tube, it was gradually withdrawn so that the soft concrete might flow out against the surrounding soil and thus completely fill the space vacated by the tube. The pre-cast concrete point remained in the ground and formed the tip of the finally completed pile. More or less difficulty was experienced with the concrete points, due to breakage in driving and to the occasional tendency for the point to become driven into the tube and so wedged that it often was withdrawn with it. To overcome this difficulty, a new type of point was devised, which consisted of a conical-shaped steel point of hollow section, split longitudinally, each of its halves being hinged to the lower collar of the tube. With this apparatus the tube was centered in the pile-driver gins, the two halves of the point brought together so as to present a conical point to the ground, and the whole was driven as in the previous case. When final penetration was reached, the tube with this attached point was withdrawn a few feet, and by means of a heavy rammer or weight, which was lowered through the tube, the two halves of the conical point were forced apart and swung outward, leaving an open passage for the placing of the concrete. The tube was then withdrawn as in the previous case, as the concrete filling progressed. This so-called alligator point, while very ingenious and usually efficient, was found to have a serious drawback in that it was not always possible to obtain the full opening desired. If the halves of the cone, or either of them, failed to open fully, the sectional area was correspondingly contracted and there was danger while pulling the tube that a part of the concrete charge inside might be drawn upward with it, leaving a void which usually would be filled by the surrounding soil, thus destroying the integrity of the pile. In order to prevent such an occurrence, these piles are now driven with a loose cast-steel point which fulfills all of the functions of the two previous types and which is left permanently in place as a pile tip.

This pile has enjoyed a considerable degree of well-earned popularity in various sections of the country, and some notable installations are recorded to its credit. While its use has generally resulted satisfactorily, there have been instances of serious failure due to one or the other of two causes. First, the breaking of the continuity of the pile by occasionally withdrawing a part of the concrete as the tube is raised, and secondly by lateral distortion and displacement due to forces introduced in the soil during driving of subsequent piles. It will be readily seen that where concrete is deposited in a 16-in. tube, unless it is an easy-flowing mixture there is always the possibility of an arching

action within the concrete mass which may cause it to adhere to the sides of the tube instead of flowing out and filling the space vacated by the tube during the process of withdrawal as intended. This difficulty can be and usually is provided against by means of careful watchfulness on the part of the foreman and inspector, or by the use of some telltale device which will indicate that the concrete is settling whenever the tube is being withdrawn. The second method of failure is not so easily provided against, nor can it be always readily detected. There appears now to be almost conclusive evidence that certain soils transmit their compression forces in a lateral direction for considerable distances. The influence on surrounding objects, in the adjacent

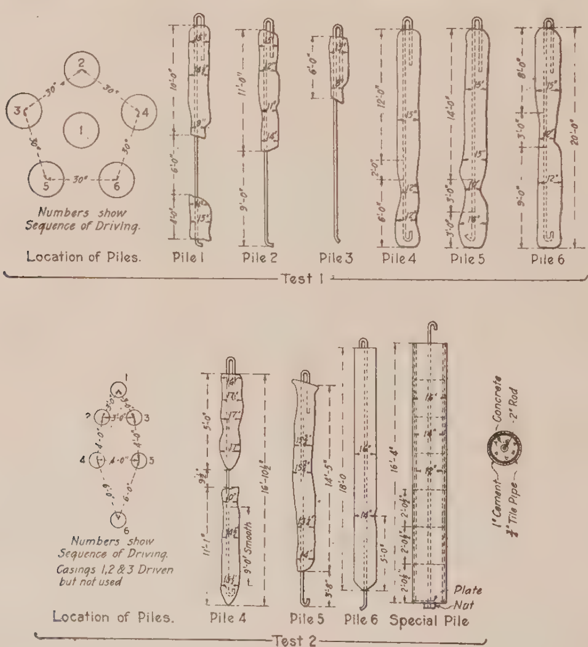

FIG. 29.—DISTORTION OF BUILT-IN-PLACE PILES.

soil which may be subjected to these forces, was well illustrated a few years ago in Cleveland, where the driving of wooden piles for the Clark Avenue Viaduct produced such compressive stresses in the soil as completely to crush an 8-ft. brick sewer although the piles themselves did not come in contact with it. Similarly, during construction of the Fish Pier, at South Boston, Mass., a few years ago, the driving of foundation piles for buildings produced a lateral displacement of an exceptionally heavy stone retaining wall surrounding the pier and retaining the pier fill. It is obvious, where such conditions exist in the soil, that the driving of several piles of this type in a group may result in serious distortion and displacement of adjacent piles already driven.

With a view to determining whether such distortion would be possible if the tubes were driven for all of the piles in a cluster before any concrete was placed, Mr. Francis L. Pruyn, of New York, conducted a test about four years ago, wherein he drove six tubes 16 in. in diameter in a cluster, the individual tubes being 3 ft. on centers. After standing 24 hours they were filled with concrete and the tubes withdrawn in the usual way. Later on, when the concrete had attained sufficient strength, the piles were pulled out of the ground by means of heavy rods which had been imbedded in the concrete for that purpose, and the appearance of the several piles as then found is shown in Fig. 29. From this test Mr. Pruyn concluded that the pressure set up in the soil by

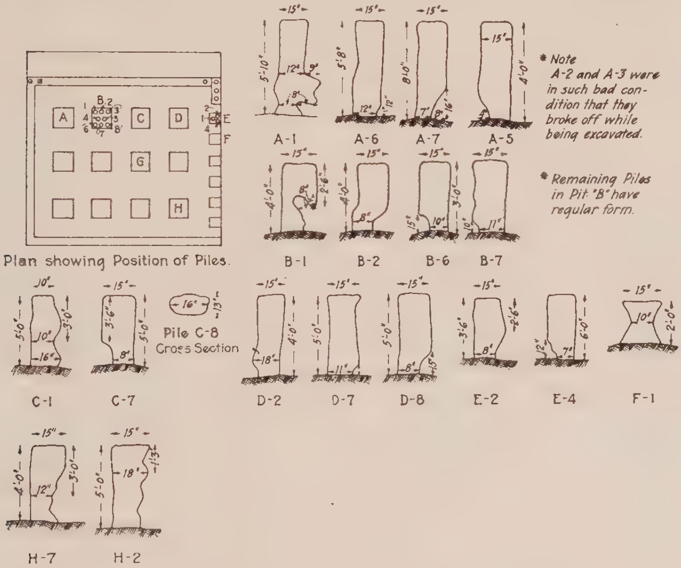

FIG. 30.—DISTORTION OF BUILT-IN-PLACE PILES.

the driving of the tube remained active even after a lapse of 24 hours. A similar result from that cause is shown in Fig. 30.

Fig. 31 illustrates the condition of some of these piles as disclosed by excavations made under the foundation of a building in Chicago a few years ago. The settlement of the building required that it should be underpinned to prevent further damage. This was done by means of open circular wells sunk to hard bottom and located under the building walls. In the sinking of these wells many of the previous piles were uncovered, and their condition is clearly indicated in the illustration by the longitudinal elevations and frequent transverse cross-sections of the well excavations.

Notwithstanding these instances, it must be admitted that with the exception of a few such cases which have occasionally come to light, this type

of pile has proved uniformly successful and efficient in thousands of installations and under a great variety of conditions, and that test loads which have frequently been applied have usually indicated a remarkable sustaining power. Likewise piles of this type have been uncovered at various times by adjacent excavations and found, generally speaking, to be perfect in shape and condition. It seems likely, therefore, that various soils act differently in respect to the transmission of compressive forces, and it is also reasonable to suppose that many deformed piles are successfully carrying considerable loads without any indication of their condition. In fact, the very compression of the soil which brings these conditions about may itself be a factor in furnishing

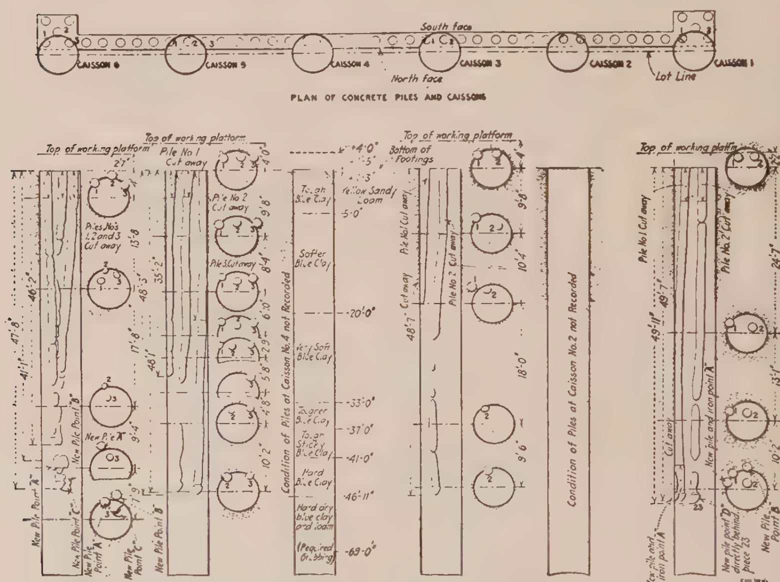

FIG. 31.—CONDITION OF BUILT-IN-PLACE PILES DISCLOSED BY EXCAVATION.

adequate support exclusive of the piles, due to its resultant compactness. The rate of progress in the driving of "Simplex" piles is retarded somewhat by the fact that the pile driver must be utilized in pulling the tube during process of concreting, and thus work can proceed on only one pile at a time with a given machine. As the tube is withdrawn, its upper end is elevated so as to require the hoisting of the concrete before depositing.

Very severe strains are brought to bear upon the pile driver because of the enormous power required to start the tubes after they have been driven. Wear and tear on these machines is, consequently, very great, and it is not uncommon to lose a large amount of time on account of repairs to machine and apparatus.

The Pedestal Pile.—The "Pedestal" pile (Fig. 32) was invented in 1909 by Mr. Hunley Abbott, and combines the essential features of the "Simplex" pile with somewhat distinctively individual qualities. Instead of adopting a point on the tube, the "Pedestal" pile employs a solid plunger which occupies the interior of the tube and projects a few feet below the bottom thereof. This plunger is pointed at the lower tip and its top end, and is provided with a collar which rests upon the upper edge of the 16-in. tube. The point, tube and plunger are driven as a unit by an ordinary pile driver. The plunger is then withdrawn, leaving the open-ended tube in the ground with a recess in the soil below. In this recess, concrete is deposited and the plunger is again used, this time as a rammer to force the concrete against and into the surrounding soil with such force that a bulb-shaped footing is formed upon which the body of the pile rests. The tube is withdrawn and concrete deposited as in the case of the "Simplex" pile, the result being a 16-in. cylindrical column

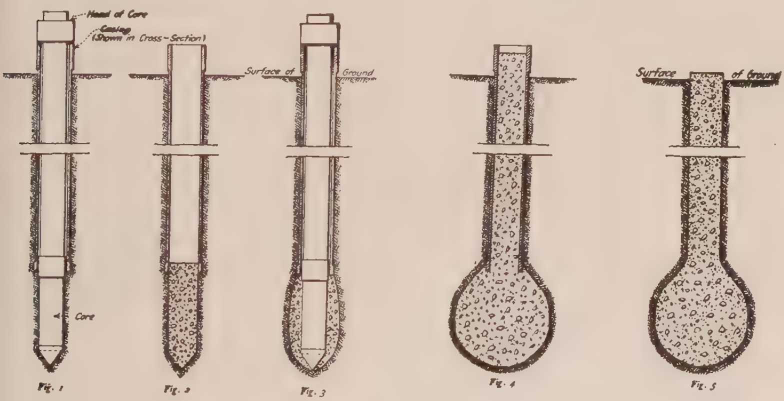

FIG. 32.—PEDESTAL PILE.
(Built in place.)

resting upon an enlarged base. This type of pile obviously possesses all the advantages of the "Simplex" pile and in addition an increased supporting power due to the spread footing. In other words, if the "Simplex" type of pile is employed to support a given load, the addition of the bulb-shaped footing will afford that much additional security against settlements. On the other hand, the character, shape and dimensions of the bulb which is formed are matters of speculation and not of certainty. In some soils it is doubtful if a bulb of any appreciable size is formed. In other cases, if the resistance of the soil for any reason is not uniform, the shape of the bulb will follow the direction of least resistance. If the pile is properly driven and the stem retains its integrity, the bulb footing is, so far as serving any useful purpose, superfluous. It must be conceded, however, that, in some classes of soft soils which are of a depth in excess of the ordinary available pile length, this bulb footing may be very valuable as an aid in safely distributing the

load. This pile naturally possesses all of the disadvantages of the "Simplex" type, especially that due to possible distortion and displacement of its stem.

A group of "Pedestal" piles driven at Long Island City, N. Y., was uncovered by direction of the Building Department, to permit an examination to be made of their condition. This was found to be excellent in every respect. It is reported that the material in which they were constructed consisted of rubbish fill, muck and sand, the piles being 20 ft. long. It is apparent in this instance, at least, that the rubbish and muck were sufficiently compressible to permit of being displaced laterally without setting up pronounced elastic forces in the soil.

Piles of the "Simplex" and "Pedestal" type cannot be used in semi-fluid material, or in situations where a portion of the length must be constructed in water, because the concrete filling would escape laterally when the tube was withdrawn. It is possible, however, by a slight modification of the method, to meet these conditions when necessary. The heavy driving tube is put down in the usual manner and, after some concrete has been deposited, a light steel shell may be inserted in the tube, filled with concrete and left permanently in place to retain the fresh concrete after the driving tube has been withdrawn.

Peerless Pile.—Another type of built-in-place pile which has recently been introduced is known as the "Peerless" pile (Fig. 33). Short lengths of thin reinforced concrete hollow shells are superimposed upon one another to the approximate length of pile desired. These shells are assembled upon a cast-steel point or shoe, and a heavy plunger or follower is inserted through the hollow interior so as to rest upon this steel shoe. On the upper portion of this plunger is an adjustable collar which overlaps and bears upon the top section of the concrete shell. This combination of shells, point and plunger is then driven to refusal by means of an ordinary heavy pile driver. It will be seen that the steel point receives the direct driving impact which is transmitted by the plunger and that the concrete shells are merely followed down without being called upon to take any strains other than those necessary to overcome their own surface friction against the surrounding soil. The maximum section of the cast-steel shoe is of a slightly larger diameter than that of the

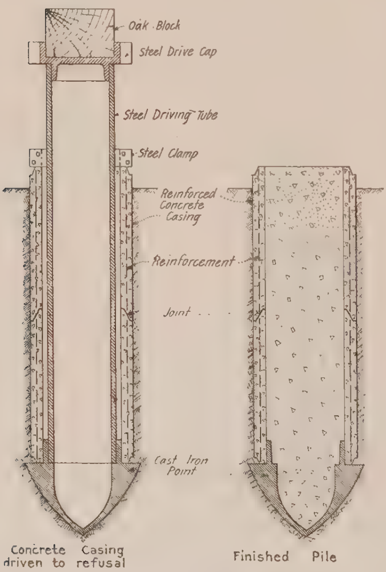

FIG. 33.—PEERLESS PILE.
(Built in place.)

concrete shells, so that the former produces all of the displacement in the soil, while the opening made by it, being slightly larger than the shell, minimizes the amount of surface friction against the latter. When the shells have been thus driven, the plunger is withdrawn and the hollow interior of the shells filled with concrete. The reinforced concrete shells serve the purpose of a driving tube and also eventually form a part of the pile itself. This type of pile has not been used to an extent which would warrant an expression as to its relative advantages or disadvantages. It would seem that in the case of a water-bearing soil, trouble might occur from infiltration of ground water through the several joints between successive shells. There would appear to be no good reason, however, why these joints should not be made reasonably water tight by the insertion of some form of water-tight gasket. It also seems likely that the cost of preparing the shells might render such piles uneconomical in comparison with other available types. The general principles utilized, however, appear to offer some advantages over other built-in-place types. There is no lost time in pulling shells, neither is the concrete required to be elevated during the process of filling, as in the case of the two previously mentioned types. It is possible to inspect the interior at all times during the filling, and ample strength is provided by the shells against distortion due to lateral pressure. Generally speaking, it will also be possible to construct piles of a greater depth by this method than with the "Simplex" or "Pedestal" types, in which the difficulty of withdrawing the tubes limits the possible length of the pile to not over 50 ft. In the case of the "Peerless" pile, it is only necessary to use a plunger of sufficient length and to build up sections of shell to the extent required.

Tubular Piles.—Tubular piles have been used for a great many years for various purposes, but their use did not become common until fifteen or twenty years ago, when they were introduced to a considerable extent for the solving of some foundation problems in the city of New York. This use seems to have originated in the patented method adopted by Breuchaud in 1896, wherein he utilized them in the underpinning of buildings in and about New York City. The process consisted of cutting a narrow, vertical breach in the masonry of the building wall and inserting in this slot a short section of 12-in. steel tube, which was then forced vertically into the ground by means of hydraulic jacks of 60 to 100 tons capacity, reacting against short horizontal girders built in the masonry above them. Successive sections of double strength 12-in. pipe were coupled together so as to extend the length of tube until it finally reached hard bottom. The earth was then washed from the interior of the tube and concrete deposited until the pipe was filled. Bearing plates were adjusted at the top of the pipe and the load of the building transferred to them. The usual progress made by this method was about one foot per hour.

Later on, the use of "Tubular" piles became common in the construction of foundations for new buildings. In general, the method has been to drive these piles in sections sometimes to a total depth exceeding 100 ft., or until they reached ledge, and to insert therein steel rods which were incased in the concrete filling of the tube and rested at their lower extremity upon the

ledge. It has been customary in New York City, where this method has been extensively employed, to figure these piles as columns fully supported laterally, with an allowance of 500 lb. per sq. in. on the concrete section and 6000 per sq. in. on the cross-sectional area of steel, including that of the pipe itself. The steel rods used for this purpose are usually 2 to $2\frac{1}{2}$ in. in diameter, and as many as 117 tons have been supported upon a single 12-in. tubular pile 70 ft. in length. The tube employed for this purpose is usually 12 in. in diameter and $\frac{3}{8}$ in. in thickness. In order to transmit the enormous load carried, it is necessary to square carefully the abutting ends of successive sections of pipe and reinforcing rods, and, in order to avoid excessive external friction during driving, interior sleeves are adopted (Fig. 34). Heavy pneumatic hammers are usually employed in the driving of the piles, being handled by construction derricks. When the tube has finally reached ledge, its interior is thoroughly washed out by means of a strong water jet, but frequently in later practice by a stream of high-pressure compressed air which often proves more efficient in removing all soil from the interior of the pipe.

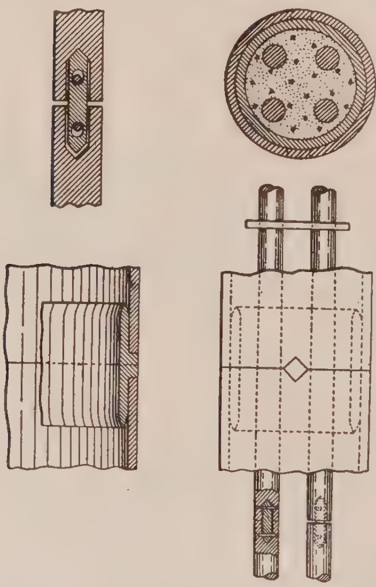

FIG. 34.—SPLICES IN STEEL TUBES AND REINFORCING RODS FOR TUBULAR PILES.

It is sometimes urged against the use of "Tubular" piles, or other types having a permanent steel casing, that the steel pipe will ultimately be destroyed by rust. The writer has been interested in this subject for a great many years, and has reached the conclusion, after observing the condition of hundreds of specimens of steel and iron embedded in the soil for long periods, that there need be no apprehension of trouble from this source. The oxygen needed for the rusting process is to a very considerable extent excluded from the soil, and even though there be a sufficient amount to produce some oxidation, the first layer of rust which forms being prevented by the surrounding earth from dropping off, serves as a protection to the remainder of the metal against any progressive rusting action. It will almost invariably be found, where wrought, steel or cast iron is uncovered after several years' burial in the ground, that the outer layers of rust have amalgamated with the adjacent soil and have formed what is equivalent to a rust joint which thoroughly protects the body of the metal.

Compressol Piles.—The "Compressol" pile (Fig. 35) is of French origin and has enjoyed a considerable use on the European continent. It has been little used in this country, however, although it was introduced in 1913 at Perth Amboy, N. J., in the construction of some building foundation work at that place. The process consisted in elevating a massive conical-shaped weight, 8 ft. long and 3 ft. in diameter, weighing $2\frac{1}{2}$ tons, with its point suspended downward. This weight is then dropped so as to penetrate the ground at the point where the footing is to be constructed. Successive blows by this instrument eventually produced a cavity which extends to the desired depth

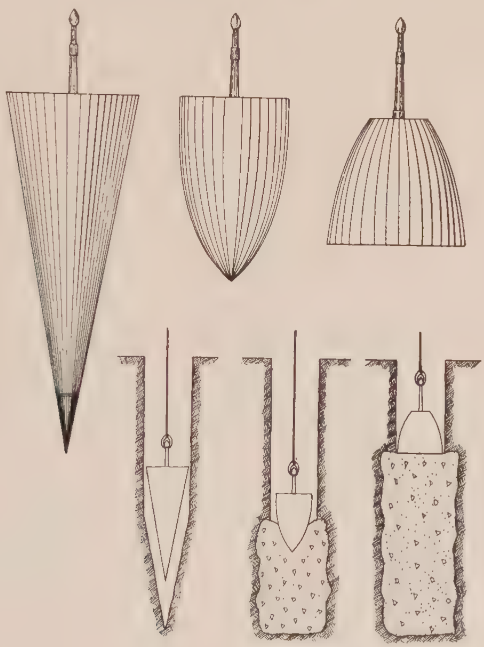

FIG. 35.—COMPRESSOL PILE.
(Built in place.)

or bearing stratum. If the material constituting the particular soil is water bearing, clay and cinders are dumped into the cavity and rammed by the succeeding blows into the surrounding soil so as to render it impervious. When the desired depth has been reached, concrete is deposited in the bottom of the cavity and by means of a somewhat more rounded tool with an olive-shaped point, weighing about two tons and similarly used, the concrete is expanded laterally into the surrounding soil, giving a spread to the footing. Eventually the entire cavity is filled in this manner, leaving an irregular-shaped column of concrete in the soil. Large rocks are often used to con-

solidate the concrete mass and to assist in displacing the surrounding soil. While this process is apparently a simple and economical one, it suggests many elements of uncertainty as to final dimensions and conditions of the resulting pile. Nevertheless it may be well suited for many temporary purposes where the element of cheapness is paramount to that of known efficiency.

Wilhelmi Pile.—Another type of pile little heard of in this country is the "Wilhelmi" pile (Fig. 36). This pile is of French invention and resembles very closely the "Pedestal" pile in final result, except that the bulb chamber is formed by exploding a charge of dynamite in the lower portion after the tube has been driven.

The Caisson Pile.—The "Caisson" pile (Fig. 37) appears to have originated in Chicago in the year 1893. In reality it can hardly be classed as a pile, but

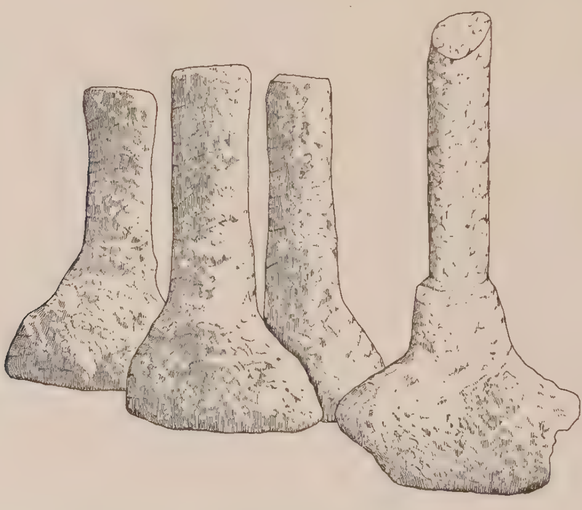

FIG. 36.—WILHELMI PILE.
(Built in place.)

resembles more closely an ordinary pier footing except that the excavation is made to the exact dimensions of the pier, and concrete forms, therefore, are not required. The excavation is circular in cross-section and of sufficient area to support the pier load in compression. This circular excavation is carried down to good bottom, the walls of the excavation being lined in short sections by wooden staves held in place by interior rings. When satisfactory bottom is reached, the excavation is enlarged in the shape of a frustum of a cone, so that the load may be safely distributed over a sufficient area. The excavation is carried on by the pick-and-shovel method, small-sized buckets being used to elevate the earth as it is loosened. When the excavation is completed, the bottom may be inspected and concrete then deposited, first filling the footing chamber and later the circular shaft. The sections of wooden lining,

together with their supporting rings, are removed as the concrete filling progresses, so that the final construction results in a circular concrete column resting upon a spread footing and capable of supporting safely a total pier load of an amount depending upon the dimensions of the shaft and footing. It is possible for a man to excavate within the limits of a 3-ft. diameter shaft. This shaft when filled with concrete is capable of supporting a 250-ton load, using a unit compressive value on the concrete of 500 lb. per sq. in. If loads in excess of this amount are to be applied, the diameter of the circular excavation must be increased accordingly. The area of the base of the enlarged footing must be made sufficiently greater than that of the shaft to distribute the total load safely upon the supporting soil. This method has been used in the soft Chicago clays to depths of 80 ft. or more. Oftentimes it becomes necessary, in passing through water-bearing sands and other unstable soils, to use a light vertical shield which retains the shape of the hole while the wooden lining is assembled inside of its protecting shell. In some instances it becomes necessary to adopt the pneumatic process in the sinking of these piles, in order to penetrate troublesome material. The writer has had occasion to use this system to a considerable extent in the vicinity of Boston, Mass. On account of the prevalence of water-bearing strata in that district, the temporary lining adopted has generally been of steel cylinders, either driven in lengths or made sectional and bolted together.

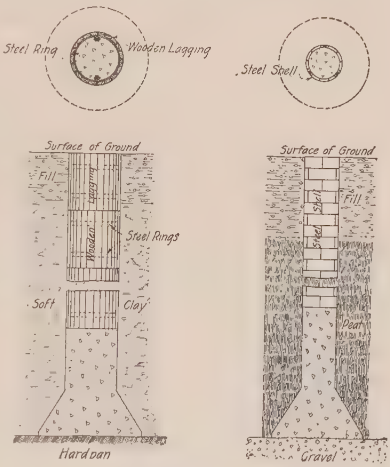

FIG. 37.—CAISSON PILE.
(Built in place.)

Wherever considerable loads are to be supported at given points, the "Caisson" pile will usually be more economical than equivalent clusters of individual concrete piles. Generally speaking, however, this does not apply to loads under one hundred tons. It is not possible to construct "Caisson" piles under all conditions, and, therefore, clusters of individual piles may frequently be necessary in any event. Since the "Caisson" pile usually rests directly on top of the bearing soil stratum, its length is materially less than that of driven piles for the same location, which necessarily must enter a sufficient distance into the bearing soil to develop the necessary frictional resistance.

COMBINATION PILES.

Various combinations of concrete and wood piles and of pre-cast and built-in-place piles have been used under special circumstances. Tubes

driven by the "Simplex" method are sometimes utilized to receive pre-cast piles which are merely inserted inside the tube after the latter has been driven and a small amount of concrete deposited above the point. The pre-cast pile is forced into this fresh concrete, and grout is poured into the space between the pile and the tube. The tube is then withdrawn in the usual manner. This arrangement avoids the necessity of subjecting the pre-cast pile to any driving strains and prevents any distortion of the pile when the tube is pulled.

An interesting instance of combining the two systems is reported in the case of the 25-in. diameter concrete piles for the Atlantic City Music Hall Pier. These piles were cast with bulb-shaped footings 3½ ft. in diameter, and were sunk into place through the sand by the jetting process. In order to avoid the handling of such extremely heavy piles, only the bulbs and a short length of the stem were pre-cast, the reinforcing rods, however, projecting for their full length. A light steel shell of 25-in. diameter surrounding the reinforcing rods was attached to the pre-cast portion of the pile and the

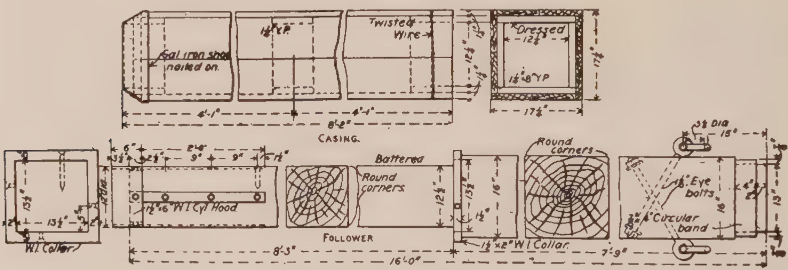

FIG. 38.—FORM AND BOX FOLLOWER FOR COMBINATION WOOD AND CONCRETE PILE.

joint with it made water tight. After the footing with the attached shell had been jetted into place, the remainder of the pile was cast in place by merely filling the shell.

In the construction of the Delaware, Lackawanna & Western Hoboken Terminal, in 1905, the station platforms were supported upon wooden piles whose tops were driven 8 ft. below grade by the following means (Fig. 38). A square box consisting of 2-in. planks 8 ft. long was constructed upon the head of each pile. A wooden follower was inserted through this square box and rested upon the head of the pile. By means of a collar near the top of the follower, the square box was driven into the ground with the pile. The follower was then withdrawn and the box filled with concrete, thus insuring safety against decay of the wooden pile heads.

In another instance, hollow steel pipes surrounded wooden pile heads and permitted the entrance of a follower so that the pile head might be successfully driven below the water level, after which concrete was deposited inside of the pipes.

Wooden piles for wharves and piers have often been driven in the usual

manner, and the top portion standing in the water surrounded with concrete shells, the intermediate space being filled with grout or concrete.

The "Ripley" pile, so called, consists of a wooden pile wrapped with a metal mesh and concrete so as to form a concrete pile with a wooden core.

Some simple expedients are frequently resorted to in special cases where economical considerations preclude the adoption of more elaborate or expensive types. For example, common auger borings up to 15 in. in diameter have been successfully made in gumbo soils and the resulting hole filled with concrete, thus forming a reasonably satisfactory type of concrete pile for moderate loads. In another instance, grouting the underlying soil was resorted to, working from the bottom upward, thus consolidating a column of otherwise unsatisfactory soil immediately under the load.

Still another method of securing concrete pile foundations has been to build and jet the piles simultaneously, using very light sheet-steel shell sections of No. 20 gage, telescoped together similar to ordinary stove piping, a $1\frac{1}{2}$-in. jet pipe being introduced in the center and the shell filled with concrete. By applying the jet pressure, the soil is washed away in advance of the pile, which drops into the cavity from its own weight. The writer has sunk piles to a depth of 80 ft. by this process, in fine sand and silt.

Selection of Type of Pile.

The determination of the particular type of concrete pile which should be adopted for a given case usually presents more or less difficulty. The problem may generally be solved, however, by a careful consideration of all the factors physical and economical which enter into the situation.

In the construction of wharves, docks and other works of a class which requires that a portion at least of the pile length shall stand in or above a body of water, the choice ordinarily will be limited to the pre-cast type of pile (Figs. 39, 40, 41 and 42). In a congested locality in the heart of a busy city, on the other hand, where construction space is not readily available, the built-in-place type of pile will be more likely to answer the requirements of the case.

Similarly, if it is necessary, because of important considerations, to commence construction operations immediately, and especially if there be no satisfactory means available for curing the piles by steam, it will usually be advisable to abandon the pre-cast pile suggestion because of the prohibitive length of time which must elapse after their manufacture and before they can be handled and driven.

The use of the pre-cast pile is often undesirable when there is not readily available a sufficient source of water supply to furnish an adequate volume for the jetting process. The amount of water used in jetting piles is, in the aggregate, considerable. From 100 to 200 gals. per minute is probably not an excessive estimate of the quantity of jet water required in sinking piles by this method, and, where such volumes of water are not at hand, economical considerations may suggest the adoption of built-in-place piles in preference to those which require jetting.

Sometimes it is necessary to drive only a comparatively small number of concrete piles for a given purpose, in which case the expense of transporting, erecting and removing the extremely heavy driving apparatus necessarily employed with built-in-place piles will render the cost of such a small group prohibitive. In such cases pre-cast piles may be driven or jetted with the aid of the construction derrick, utilizing a steam hammer suspended from the derrick boom, or a small timber driving frame (Fig. 43) may be employed. Where there are but a few piles required for a given installation, the writer has frequently recommended the use of light tubes in the form of second-hand wrought-iron pipe driven into the ground by some simple apparatus, after which they may be washed out and filled with concrete, leaving the wrought-iron shell as a component part of the pile. This, of course, is merely an adaptation of the tubular pile previously described.

If it is deemed desirable to adopt the pre-cast type of pile, the particular conditions surrounding the work may suggest the adoption of one or another of the patented types of pre-cast piles, although in general any type of pre-cast pile can successfully be driven if proper facilities are available. If the material through which the pile is to be driven is of a nature which renders jetting more or less ineffective, recourse must be had to long-continued hard driving. In such a case, a pile of the "Cummings" type probably offers as good a solution as can be found, especially for long lengths of pile. The "Giant" pile also is obviously well suited for such a condition, although it seems probable that its use must be limited to lengths not exceeding 25 to 30 ft. In situations where the use of the hammer is difficult, expensive or inconvenient, the "Bignell" type of pile may offer a solution, relying upon the jetting process only. In any of these cases, an unpatented type of pile may be designed and driven with the use of proper cushion caps and hammers so that final results may be equally satisfactory. The question involved, therefore, is merely one of relative economy.

With regard to the several types of built-in-place piles, it seems reasonably clear that those types which leave their shells permanently in place, such as the "Raymond," "Peerless" and "Tubular" piles, offer a somewhat better guarantee of the final and continued integrity of the completed pile. The use of the "Raymond" pile, however, should probably be limited, as previously suggested, to those cases in which a substantial portion o the length of pile will rest in firm soil in order that too great a proportion of the load will not have to be carried on the small sectional area near the point. These three types of piles may also be driven to greater depths than other types of built-in-place piles for the reason that there is no tube to be withdrawn, a process which requires enormously powerful tackle and lifting apparatus, especially when the length of tube is great.

In the great majority of cases, however, built-in-place piles of the "Simplex" and "Pedestal" type may be used with perfect safety and oftentimes with decided economy. It seems probable to the writer that in soils which are definitely and permanently compressible, such as peat, silt and rubbish filling, there is little likelihood of a subsequent reaction of a nature which will cause damage to the piles. On the other hand, it appears reason-

FIG. 39.—PRE-CAST PILES FOR WHARF.

FIG. 40.—WHARF CONSTRUCTION ON CONCRETE PILES.

FIG. 41.—WHARF CONSTRUCTION AT CHARLESTON, S. C.

FIG. 42.—WHARF CONSTRUCTION AT CHARLESTON, S. C.

ably conclusive that some of the clays, sands and gravels are decidedly elastic in character under certain conditions, and that the resultant action due to driving piles into them is one of displacement laterally rather than of simple compression. This displacement apprently sets up elastic forces of considerable amounts which react against the soft concrete after the protecting tube is withdrawn, tending greatly to displace and deform the concrete sections and also to disturb the adjacent piles, previously driven, because of the lateral motion imparted to the surrounding soil. When this type of pile is used in soil which is not known to be positively compressible, it will be well to avoid driving successive piles within six feet of one another. This require-

FIG. 43.—PILE BEING DRIVEN WITH AID OF CONSTRUCTION DERRICK AND TIMBER DRIVING FRAME.

ment undoubtedly places a considerable economic burden upon the driving contractor because of the greater amount of machine moving required. Nevertheless, unless actual inspection may be had of test piles driven at closer intervals and indicating an entire absence of damage by deformation, the owner is not justified in sanctioning the more economical procedure.

Where heavy concentrated pier loads are to be carried on piles, it will usually be well to give consideration to the possible adoption of the "Caisson" pile method. In order that this type should be applicable to a given case, the soil at the base must be of a nature which will permit chambering in the shape of a frustum of a cone, and this implies a considerable degree of firmness and cohesion between the particles constituting the soil. There will usually be

no difficulty in chambering enlarged footings in dry clay, stiff mud or peat. This method will obviously be impractible in a loose soil such as sand, gravel or semi-fluid mud. This type of pile will usually not be economical in comparison with other types for loads less than one hundred tons each.

It is probable that the combination wood and concrete pile might more often be used than is customary. Where the conditions impose only a very moderate load upon each pile, and when large numbers of piles are required in the aggregate, a very considerable economy will result if the wooden pile is used in conjunction with a hollow pipe or box follower, so as to permit the head of the wooden pile to be driven below ground water level while the follower portion is filled with concrete.

RESULTS OF SOME PRELIMINARY TESTS ON THE EFFECT OF HYDRATED LIME ON MORTARS AND CONCRETE.

By H. H. Scofield* and M. J. Stinchfield.†

The writers are cognizant of the fact that many engineers are of the opinion that the presence, in small quantities, of hydrated lime in concrete has a beneficial effect. Numerous text-books and other publications have cited the advantages thus to be gained but in most cases, attention has been directed to the fact that the hydrated lime admixture can be used advantageously only in lean mixtures and not in the richer ones.

There has been a tendency in recent practice to use hydrated lime in the richer mixtures in order, as has been said, to increase the plasticity, the density, and the ease of handling, to lessen the tendency to segregation and reduce the number of shrinkage cracks.

The tests herein reported constitute the results of some preliminary series to determine the effect of hydrated lime when used in these comparatively rich mixtures. The tests comprise the determination of density, strength, absorption, and resistance to abrasion of mortars with and without a hydrated lime admixture. Two series have been made to determine the strength and relative contraction and expansion of concrete with and without lime content.

The tests were made by the writers in the Laboratory for Testing Materials, Purdue University.

Materials Used in the Tests.

The following materials were used in the tests. Universal Portland Cement was used throughout all tests except the density determinations. In these, the cement was a mixture in equal parts of Alpha, Atlas, Lehigh, Speed, Universal and Wabash.

Hydrated lime donated by the Mitchell Lime Co. was used for all the tests except the density tests. For these Lion Brand of hydrated lime procured on the local market was used.

The aggregate used comprised river sand from Indianapolis; a local bank sand and gravel; Logansport limestone screenings, procured from the city engineer of Lafayette; and crushed limestone from the Monon quarries. The artificially graded sand for the density determinations was a local bank sand carefully screened and recombined according to the sieve analysis shown in Table 1.

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TABLE 1.—SIEVE ANALYSIS OF SANDS USED IN DENSITY TESTS.

Sieve No.	Size of Mesh (inches).	Per cent (by Weight) Passing.
3	0.2900	100.0
4	0.1850	78.5
8	0.0930	59.8
16	0.0420	40.0
20	0.0328	34.8
30	0.0220	25.0
40	0.0150	17.6
80	0.0070	8.3
100	0.0058	6.5

METHODS USED IN PREPARATION OF SPECIMENS.

It was the aim in all of the tests to permit as few variables to enter as possible except the variable of lime content, itself. To this end, quantities were always determined and measured by weight, the time of mixing was kept constant and the storage conditions for all specimens of a series kept the same. In all cases the dry materials entering into a batch were thoroughly mixed before adding the water. The amount of water used in each case was sufficient to bring the mass to the required consistency or degree of wetness. The consistency adopted was such that after light but thorough tamping in the molds water flushed to the surface sufficient to be observed. The consistency was about the same as that given to standard Ottawa sand mortar when the proper amount of water is used.

Inasmuch as the retention of the water by hydrated lime is thought by some to have an influence on the strength and other properties, the test pieces were all cured in moist air. In general, three, and sometimes more, test pieces were made to establish a result. Each test piece of the same kind in a series was made on a different day, the test conditions for all being kept as constant as possible.

In the density determinations, the mixing was done in porcelain lined rectangular pans which could be thoroughly cleaned and permitted no waste of material. The yield of mortar was measured to 0.0001 cu. ft.

METHOD OF TESTS.

Determination of Strength.—The test specimens for strength were 2 in. cubes for all mortar mixtures and 8 x 16 in. cylinders for concrete. Tests were made with the usual spherical bearing in the testing machine and the bearing surfaces were properly capped with plaster of paris.

Absorption of Water.—The absorption test specimens were first dried to a constant weight at the temperature of boiling water. The specimens were then immersed in water for a period of 48 hours. The amount of water absorbed was expressed as a per cent of the dry weight of test pieces.

Abrasion Test.—These tests were made upon 2 in. cube test pieces on the Dorry type of abrasion machine used in the test for hardness of rock for

road purposes. The specimen is held with one of its faces in contact with a horizontal revolving iron disk. The pressure of the test piece against the disk at the beginning of the test was 1500 grams. Standard Ottawa sand was fed on to the disk to act as an abrasive agent. After a given number of revolutions the loss in weight was determined.

Expansion and Contraction.—Brass plugs were placed in the concrete at the time of molding on two opposite faces of 4 x 4 x 24 in. prisms. The plugs were bored for the inserts of the Berry strain-gage on a 20 in. gage length. Measurements of change in length could be read to 0.0002 in. direct or to 0.00002 in. by estimation of a tenth of a dial space.

TABLE 2.—TESTS OF MORTAR WITH HYDRATED LIME ADMIXTURE.
Indianapolis River Sand, $\frac{1}{4}$ in. to 00, 1 : $2\frac{1}{2}$ by weight. Age 28 days.

Lime, per cent.	Water, per cent.	Cement per cent.	Compressive Strength, lb. per sq. in.	Weight, lb. per cu. ft.
0.0	11.10	28.6	4527	149.70
5.0	11.65	28.2	3539	146.10
10.0	14.00	27.8	2529	142.45*
15.0	13.60	27.4	3093	144.20
20.0	15.05	27.0	2431	142.9

* Trifle too wet.
NOTE.—Lime percentage is on basis of weight of cement. Water and cement percentage is on basis of total dry weight of solids. All mixtures were the same consistency as to moisture except as noted above.

TABLE 3.—TESTS OF MORTAR WITH HYDRATED LIME ADMIXTURE.
Logansport Limestone Screenings, $\frac{1}{4}$ in. to 00, 1 : $2\frac{1}{2}$ by weight. Age 28 days.

Lime, per cent.	Water per cent.	Cement, per cent.	Compressive Strength, b. per sq. in.	Weight, lb. per cu. ft.
0.0	15.05	28.6	2275	140.5
5.0	16.30	28.2	2019	139.7
10.0	17.25	27.8	1874	139.0
15.0	19.00	27.4	1475	137.2*
20.0	19.10	27.0	1522	136.0

* Trifle too wet.
NOTE.—Lime percentage is on basis of weight of cement. Water and cement percentage is on basis of total dry weight of solids. All mixtures were the same consistency as to moisture except as otherwise noted.

DISCUSSION OF RESULTS.

In Tables 2 and 3 and Fig. 1, are given the results of tests of a mortar using two commercial fine aggregates. The proportions were 1 to $2\frac{1}{2}$ by weight. In each case, the lime was added to the given materials rather than replacing a part of the cement although the effect is the same to a lesser degree, inasmuch as the percentage of cement decreases in each case. It will be noted that the percentage of water necessary to give mortars of uniform consistency or degree of wetness increases with the amount of lime. The unit weight of the set mortar decreases with the addition of lime indicating a decrease of

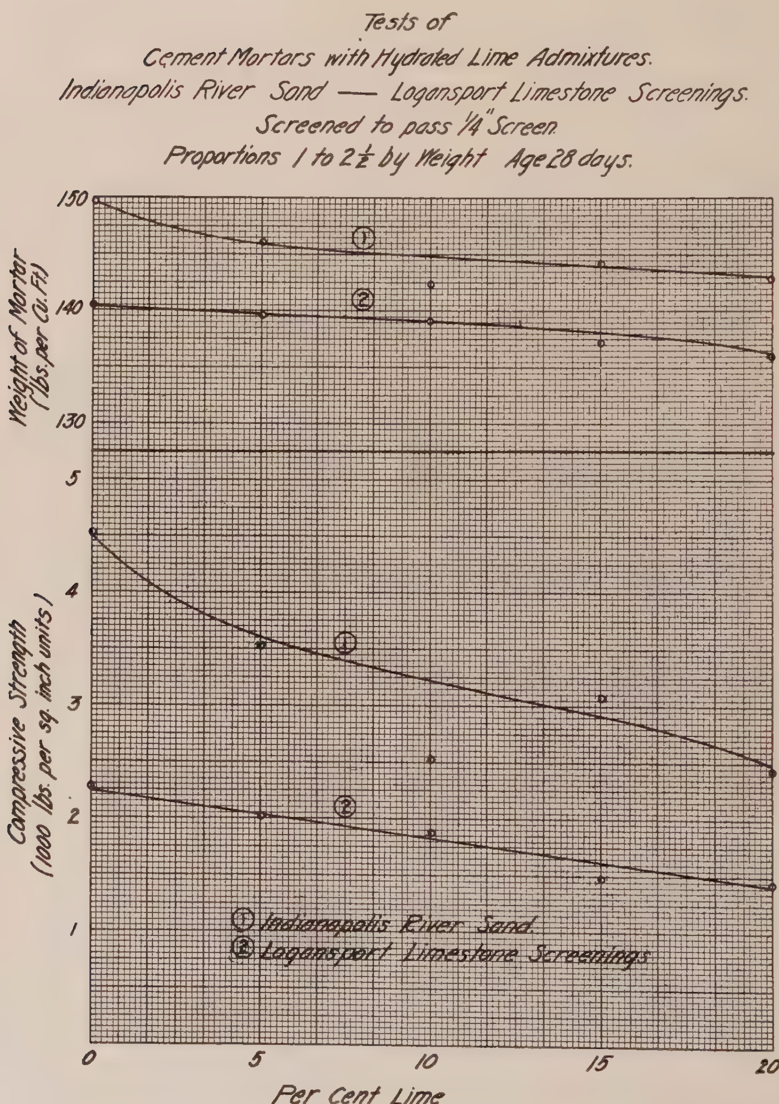

FIG. 1.—CURVES FROM TABLES 2 AND 3.

density. The loss of strength is due to the decrease in the percentage of cement and to the decreased density, both of which act to reduce the amount of cement in a unit volume of set mortar.

Tables 4 and 5 and Figs. 2 and 3 show the results of tests upon a lean mortar and a rich one, each being made of the same fine aggregate. It will be noted that in the case of both mortars, the absorption of water and the loss in the abrasion test vary inversely with the strength and density, as shown by the weight per cubic foot of the set mortar; and as might be expected the density and strength of the leaner 1 to 6 mixture reach a maximum at a 10 or

TABLE 4.

Lafayette Bank Sand, ¼ in. to 00, 1 : 3 by weight. Age 28 days.

Lime, per cent.	Water, per cent.	Weight, lb. per cu. ft.	Compressive Strength, lb. per sq. in.	Absorption, per cent.	Abrasion Test, Loss, per cent.
0	10.1	138.5	4092	6.85	7.32
5	10.5	137.5	3998	7.20	7.94
10	11.3	137.5	3982	7.89	8.19
15	12.0	136.7	3807	8.56	8.53
20	12.5	132.2	3454	9.47	10.88

TABLE 5.

Lafayette Bank Sand, ¼ in. to 00, 1 : 6 by weight. Age 28 days.

Lime, per cent.	Water, per cent.	Weight, lb. per cu. ft.	Compressive Strength, lb. per sq. in.	Absorption, per cent.	Abrasion Test, Loss, per cent.
0	10.0	132.0	1737	7.6	22.1
5	10.2	133.8	1787	7.6	19.4
10	10.45	134.8	1822	8.1	16.0
15	10.8	134.5	1895	8.3	18.7
20	10.83	134.4	1735	8.4	17.6

NOTE.—In Tables 4 and 5, lime percentage is on basis of weight of cement. Water percentage is on basis of total dry weight of solids. Water absorbed is expressed as per cent of dry weight of 2-in. cube. Loss in abrasion is expressed as per cent of original weight of 2-in. cube.

15 per cent lime content. For some reason also, unknown to the writers, the absorption increases uniformly with the lime content.

Table 6 and Fig. 4, give the results of the density determination of mortars. In order to make the results of these accurate for comparative purposes, it was necessary to use absolutely the same aggregate for each determination. The sand for this purpose was combined synthetically according to the sieve analysis in Table 1.

Four series of these were made; two in which the lime was added to the mixture in increasing percentages; one in which the percentage of cement was kept constant by adding the proportionate amount of cement as the lime was added; and one in which the added percentage of lime replaced a like amount of cement. In these results, as in the others, the percentage of

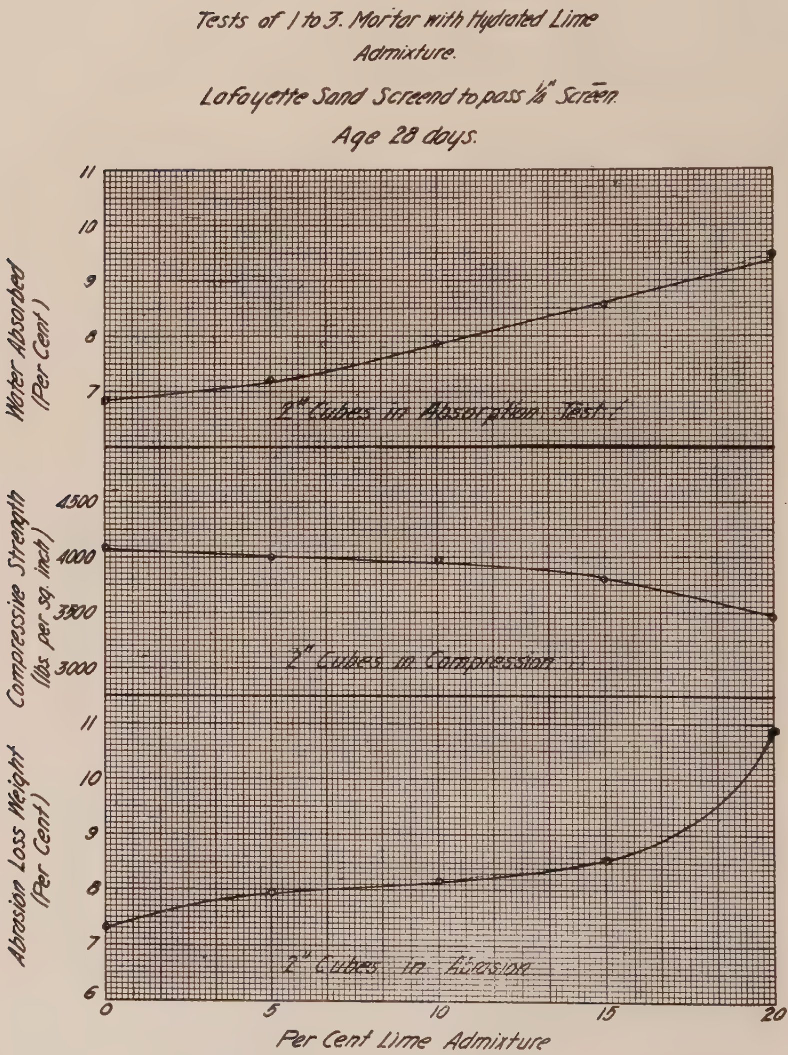

FIG. 2.—CURVES FROM TABLES 4.

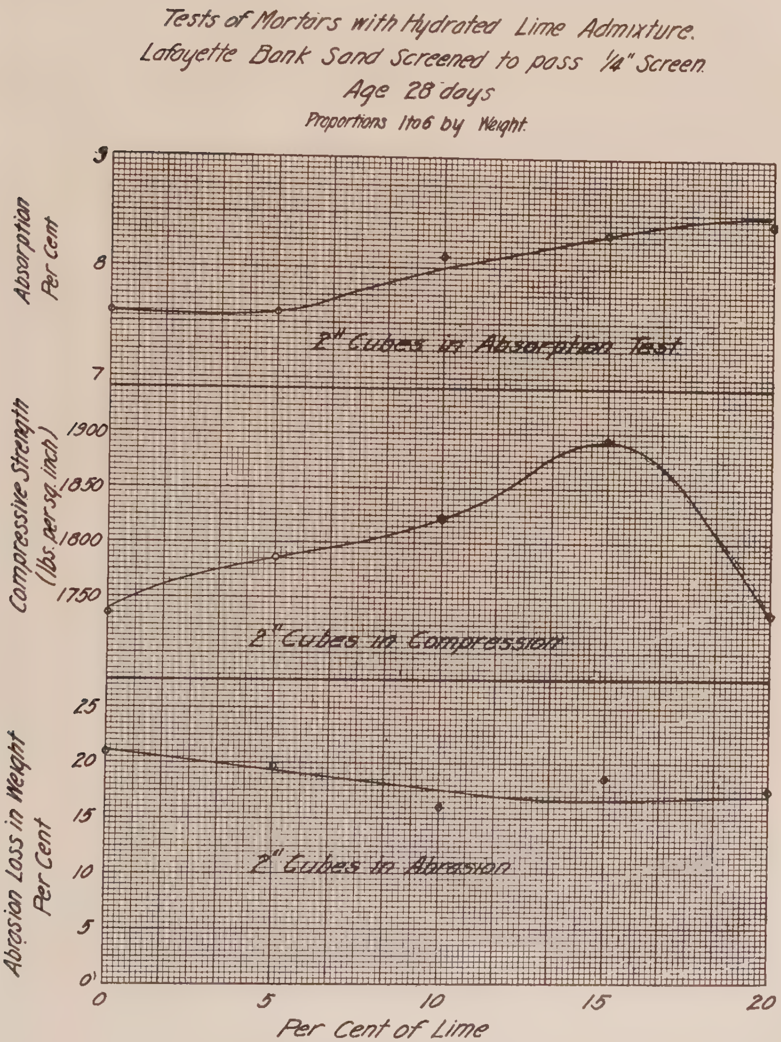

FIG. 3.—CURVES FROM TABLE 5.

water to produce a uniform consistency, increases with the percentage of lime. The computed density, which is the ratio of the absolute volume of all the solids entering into a mortar to the total volume of mortar produced, decreases with the addition of lime in all series. The reduction in density is least in the series where the lime replaces a like amount of cement and greatest in that series where the percentage of cement is kept constant by increasing its amount. It will be noted that the best criterion of strength is the amount of cement per unit volume of mortar. The absorption increases

TABLE 6.—RESULTS OF TESTS OF DENSITY OF 1 : 3 MORTARS WITH HYDRATED LIME ADMIXTURES.

Lime, per cent.	Cement, per cent.	Water, per cent.	Density, per cent.	Amount of Cement per Unit Volume of Mortar, lb. per cu. ft.	Compressive Strength, 28 Days, lb. per sq. in.	Absorption of Water, per cent, Basis of Oven Dry Weight.
Series I and II.—Lime Added to 1 : 3 Proportion by Weight.						
0.0	25.0	11.15	0.793	34.0	4392	5.54
5.0	24.7	11.55	0.788	33.4	4390	6.00
10.0	24.4	12.29	0.777	32.5	3915	6.57
15.0	24.1	13.26	0.760	31.4	3285	7.19
20.0	23.8	15.00	0.738	30.05	2695	8.33
Series III.—Per Cent Cement Constant.						
0.0	25.0	10.50	0.796	34.15	4417	6.03
5.0	25.0	11.12	0.795	34.00	4395	5.46
10.0	25.0	12.70	0.768	32.92	4040	6.94
15.0	25.0	13.10	0.752	32.25	3437	7.37
20.0	25.0	14.45	0.737	31.55	3225	8.40
Series IV.—Lime Replaced a Like Amount of Cement.						
0.0	25.00	11.1	0.793	34.0	3960	5.79
5.0	23.75	11.7	0.790	32.1	3545	5.82
10.0	22.50	12.0	0.782	30.1	3250	5.96
15.0	21.25	12.8	0.770	27.9	2880	8.00
20.0	20.00	13.8	0.754	25.65	2142	9.33

in all cases with the addition of lime. However, it must be observed that the difference in strength or absorption for a lime content up to 5 per cent and that of no lime content is not marked.

The relative expansion and contraction of concrete prisms when subjected to varying conditions of moisture and temperature, are shown in Table 7 and Figs. 5 to 7. In series I and II, a 1 : 5, by weight, concrete and mortar respectively, the change in length for all treatments is greater for the prisms with lime than with no lime. This is due to the greater absorptive power of the former when used in this comparatively rich mixture.

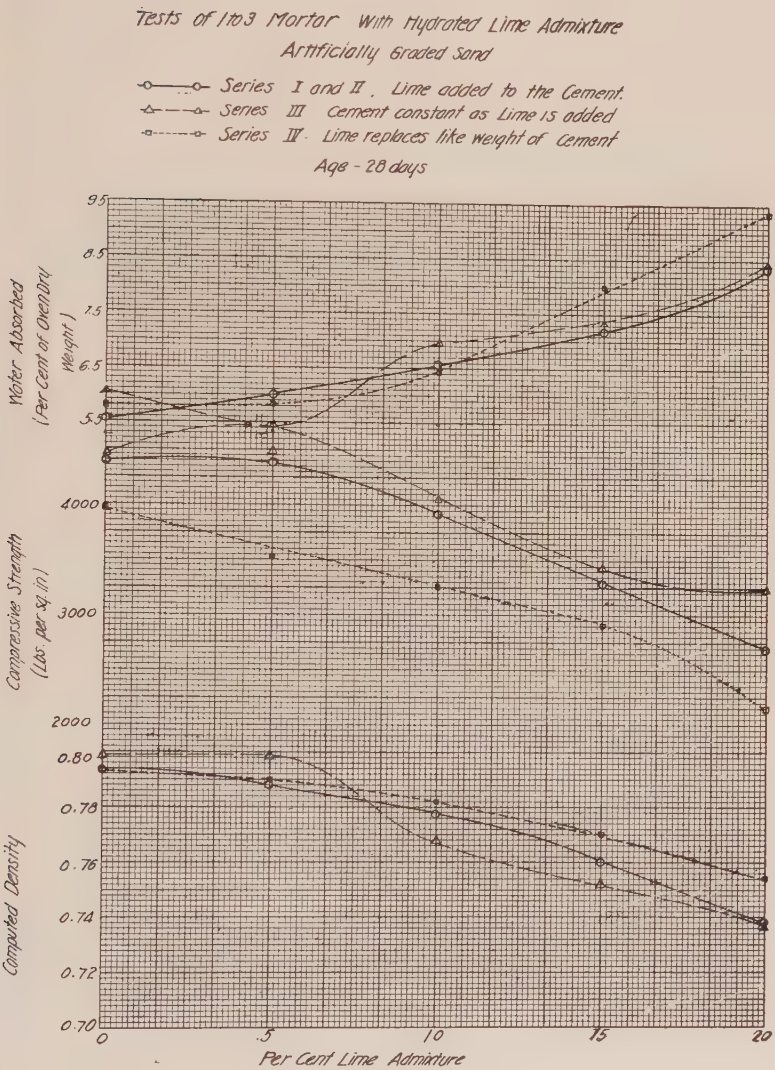

FIG. 4.—CURVES FROM TABLE 6.

Series III shows the effect of screening out the fine sand in an aggregate and then adding cement in a 1 : 5 proportion of concrete. In this case it is evident that the cement and lime are both needed to fill the voids and increase the density and strength of the mixture. It will be noted that the change in length per inch per degree is greater or less according to whether the effects

TABLE 7.—TESTS OF EXPANSION AND CONTRACTION OF MORTAR AND CONCRETE WITH AN HYDRATED LIME ADMIXTURE.

Series I.—Bank Run Gravel Concrete, Proportion 1 : 5 by weight. Age 7 weeks.

Amount of Lime, Per Cent of Weight of Cement.	Expansion or Contraction, in. per in.			Compressive Strength, lb. per sq. in.
	(a)	(b)	(c)	
0.0	+.0001725	+.0003600	−.0004725	2314
7.5	+.0001835	+.0003660	−.0005175	2170
15.0	+.0002035	+.0003895	−.0004910	2280

Series II.—Bank Sand Mortar, 1 : 5 by weight. Age 8 weeks.

	(a)	(b)	(c)	
0.0	+.0001920	+.000400	−.000553	8735
7.5	+.0002200	+.000410	−.000598	8400
15.0	+.0002225	+.000418	−.000607	6835

Treatment for Series I and II:
(a) From 71°.5 F. in inside air to 71°.5 F. in water (24 hr.).
(b) From 71°.5 F. in inside air to 100° F. in water (72 hr.).
(c) From 100° F. saturated to 10°.4 F. in outside air.

Series III.—Sand Pebbles, $\frac{1}{8}$ to $\frac{1}{4}$ in. in 1 : 5 Mortar. Age 8 weeks.

	(a)	(b)	(c)	(d)	
0.0	+.000752	−.000664	+.000377	−.000467	2160
7.5	+.000567	−.000635	+.000270	−.000502	2980
15.0	+.000692	−.000572	+.000395	−.000545	3392

Treatments:
(a) From 33°.8 F. in inside air to 180° F. in oven.
(b) From 180° F. oven dry to outside air at 18° F.
(c) From 72° F. in ordinary air to 107° F. in water (saturated).
(d) From 107° F. saturated to 18° F. in outside air.
NOTE.—In all series, positive sign indicates expansion and negative sign indicates contraction.

due to change in temperature act in conjunction with or opposition to the effects produced by the change in moisture.

Table 8 gives the results of strength tests of two series of machine mixed concrete at the age of 9 months. The leaner 1 : 3 : 6 mixture shows the expected increase of strength with the addition of lime up to 15 per cent.

The results for the 1 : $1\frac{1}{2}$: 3 mixture are unsatisfactory in that the

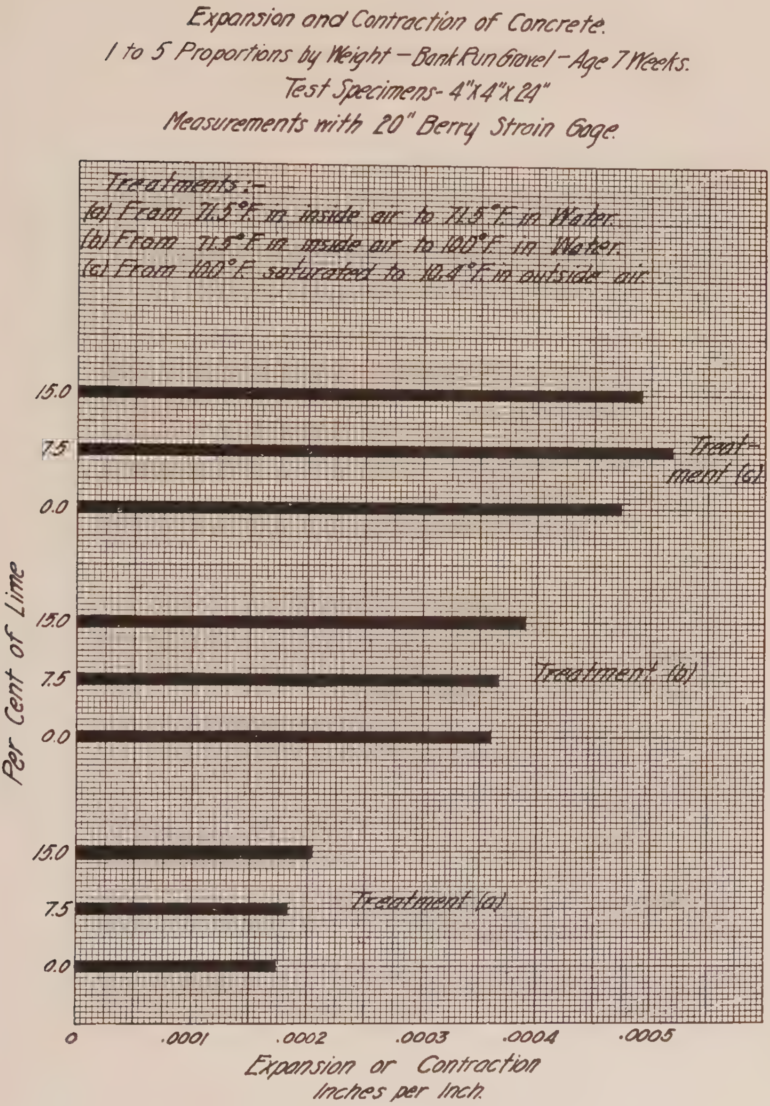

FIG. 5.—CURVES FROM SERIES I, TABLE 6.

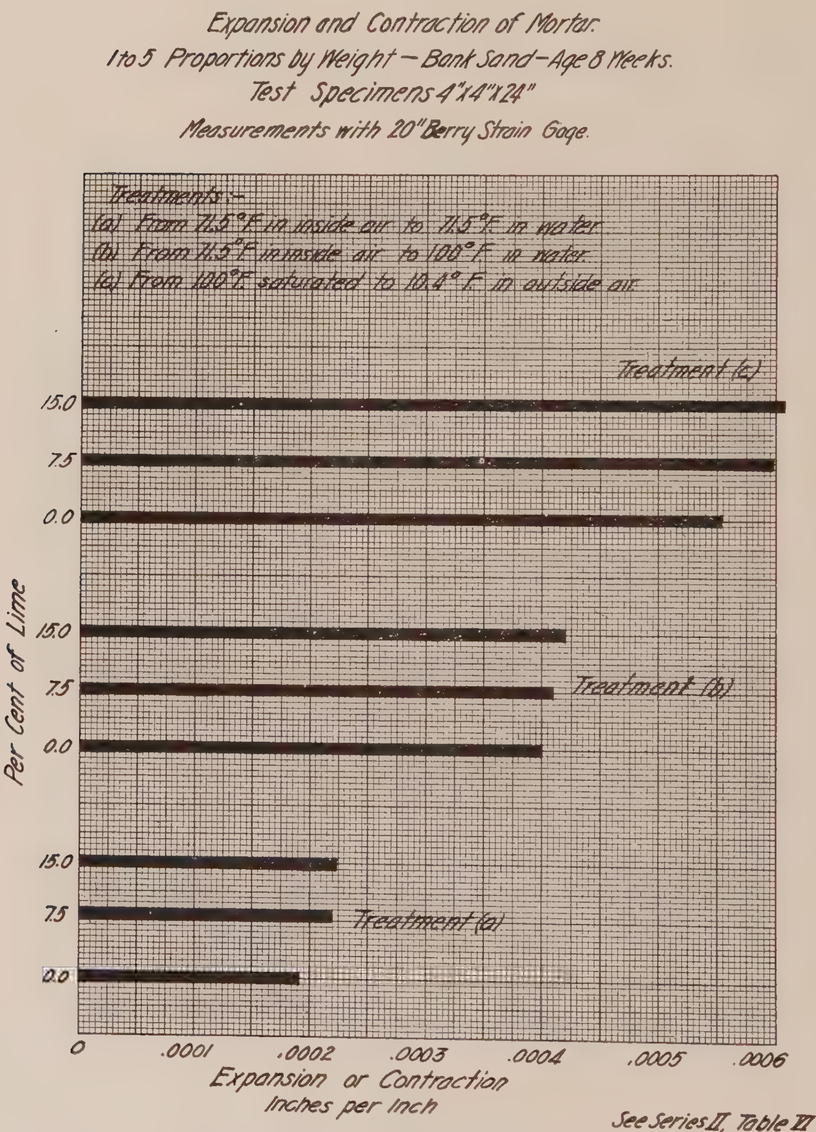

FIG. 6.—CURVES FROM SERIES II, TABLE 6.

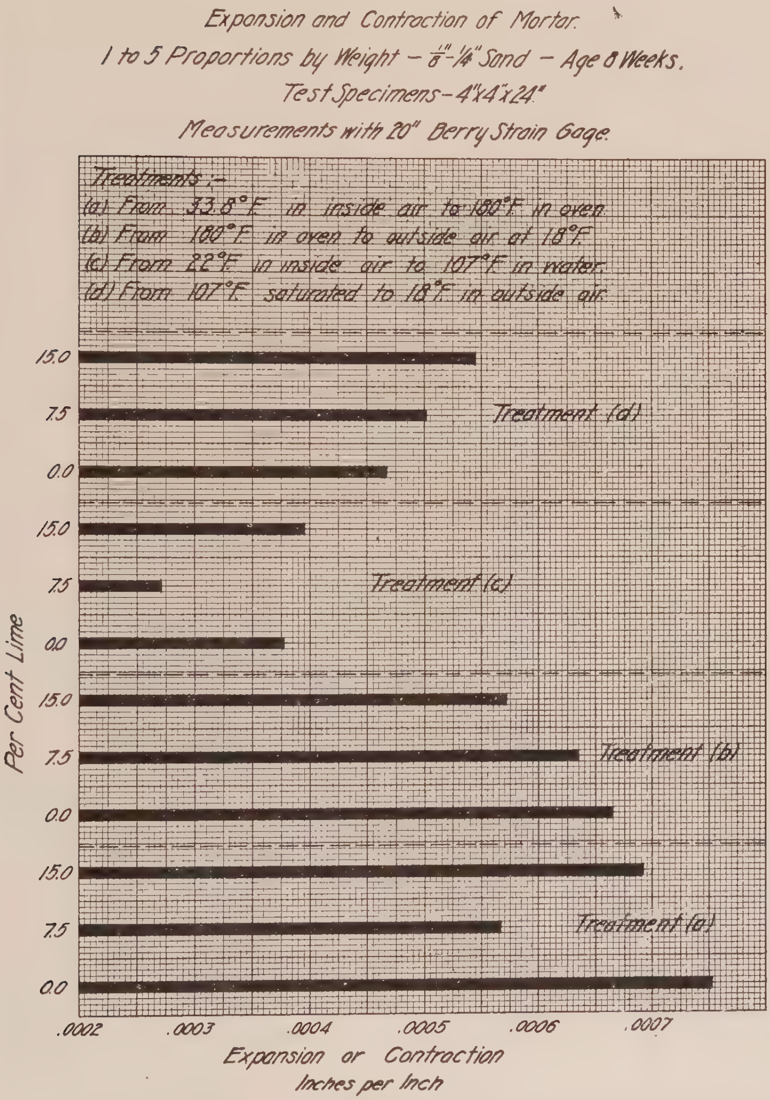

FIG. 7.—CURVES FROM SERIES III, TABLE 6.

averages show progression and retrogression for increases of lime content. The individual values gave also values showing no particular relation to the lime content. It is thought that this was due mainly to the difficulties experienced with machine mixing. It was difficult to judge the consistencies with a mixture of such an excess of fine material, and also to clean the mixer such that the batch obtained was truly representative of the ingredients put in. Future tests will in all cases be hand mixed.

TABLE 8.—STRENGTH OF CONCRETE WITH HYDRATED LIME ADMIXTURES.

Universal Portland Cement, Lafayette Bank Sand, Monon Crushed Limestone.

Proportions 1 : $1\frac{1}{2}$: 3 and 1 : 3 : 6 by volume. Age 9 months.

Per Cent. Lime.	Compressive Strength (lb. per sq. in.). 1:3:6.	1:$1\frac{1}{2}$:3.
0	984	3275
5	1090	3665
10	1416	3390
15	1610	3575

NOTE.—Lime was added to the mixture rather than replacing cement. Test specimens were 8 x 16 in. cylinders.

SUMMARY OF RESULTS.

Although it is not the intention of the authors to draw any conclusions at this time, an emphasis of the points seemingly brought out may not be out of place.

(1) Assuming that the lime in itself has no cementing value, then it must be classed with the inert material or aggregate and its addition reduces the richness of the mix.

(2) To prevent reducing the richness of the mixture, thereby lowering the strength and resistance to abrasion it would be necessary to add more cement. However, to add cement and lime both to an average aggregate which already has a sufficiency, and in many cases an excess, of fine material, reduces the density causing greater absorption of water and the attendant greater expansion or contraction.

(3) On account of the addition of a fine material like lime, a greater amount of mixing water must be used to produce the same consistency or degree of wetness. This would be true whether the material added were lime or cement or fine aggregate. This added water is in itself a cause for increased porosity or decreased density. The addition of lime must, however, act to decrease the size of the voids if not the amount, inasmuch as there are numerous tests to show decreased permeation of water under pressure through concrete with small amounts of lime present.

(4) If it can be demonstrated that a less amount of water can be used in hydrated lime concrete than with natural concrete without impairing its

workability or the efficiency of placing, a decided gain in density and strength would in all probability result.

(5) Although many of these preliminary tests have been made upon mortars because of the greater ease of handling, it remains to be demonstrated that the causes which induce certain results in a mortar would or would not give the same results when that mortar is placed in a given coarse aggregate.

The authors are at present continuing the tests to determine if possible the effect of lime additions on the segregation, slipperiness and workability of the resulting concrete and the effect of these items on the density and strength and other properties of the concrete as placed.

DISCUSSION.

Mr. Emley.

Mr. Warren E. Emley (*by letter*).—The Bureau of Standards has under way a research into the effect of hydrated lime on the properties of concrete, but the results are so far from complete that only those for the compressive strength have as yet been published. It is gratifying to learn that our results agree so closely with those obtained by Professor Scofield. It is particularly gratifying, because both of our results are diametrically opposed to certain opinions which have found wide credence among engineers.

The situation may briefly be summed up thus: Laboratory tests indisputably demonstrate that a certain fact is true, while practical experience seems to prove conclusively that it is not true. This condition seems to leave three courses open to the laboratory investigator, in order to realize the full value of his work: (1) He must prove that the conclusions which have been drawn from practical experience are erroneous, or (2) he must admit that the laboratory tests do not give the information desired, or (3) he must attempt to find the connecting link, which will enable him to interpret laboratory results into behavior on the job.

Since we have been studying this subject for some little time with this viewpoint, we may be able to throw a little light on the cause of certain discrepancies. It is claimed by many that the presence of hydrated lime in concrete will cause the retention of moisture for a long time in air storage. This moisture is supposed to produce more nearly complete hydration of the cement, and, therefore, greater strength. If this theory is correct, then it must follow that the effect of hydrated lime on the strength of concrete will be very different when the specimens are stored in dry air, or in the damp closet. In the latter case, the concrete is always saturated with moisture, so that the power of hydrated lime to retain water is not called into play. A large amount of concrete is placed above grade, exposed to the weather. The effect of hydrated lime upon such concrete can be determined only when the test specimens are stored under similar conditions—not in a damp closet.

It seems logical when making a laboratory test for absorption to begin by drying the specimen to constant weight at about the boiling point of water, as Professor Scofield has done. Only in this way can conditions be obtained which are sufficiently uniform to give comparable results. When hydrated lime is mixed with water, it forms a sticky paste. If this paste is allowed to dry in the open air at room temperature it can be rewet and worked back into the original paste rather easily. But if it is dried to constant weight by the aid of heat, it will break up into small hard granules, and will require prolonged soaking or kneading to make it take up water again. Let us consider a sample of green concrete, in which the pores are pretty well filled by gelatinous lime paste. When the concrete is exposed to the weather, it will gradually dry out, and the lime paste will shrink and solidify (at least

that part of it which does not carbonate). If the concrete were now to be immersed in water, it is conceivable that the lime could absorb the water, swell up, and fill the pores. This would give rise to high absorption and low permeability. If, on the other hand, the lime is dried out completely before the concrete is immersed, then the lime is no longer able to absorb water or to swell, and the concrete is, therefore, high, both as to absorption and permeability. The permeability is generally considered as being of greater interest to the public than is the absorption. While permeability may be found to vary directly as the absorption in laboratory tests where the specimens are dried to constant weight at high temperatures, this relation does not necessarily hold in practical work where the concrete is not so dried. It is reasonable to suppose that high absorption would be accompanied by high expansion, but the curves in Fig. 7 seem to indicate that this is not always the case. **Mr. Emley.**

It is certainly true that concrete containing hydrated lime will require more water to bring it to normal consistency than concrete without hydrate. This is true of the normal or of a similarly dry consistency. But we have evidence pointing to the conclusion that it may not be true when the consistency is as wet as that commonly used in spouting. If this is true, conclusions reached from tests of concretes made at the same consistency may be entirely reversed if that consistency is wet instead of dry.

It is frequently claimed that the consistency at which concrete will just flow down a chute is much thicker when the concrete contains hydrate, on account of the lubricating action of the lime. Although (at least according to the general belief) it takes more water to produce the same consistency when the concrete contains hydrate, this extra water need not be added, because the concrete will flow at a thicker consistency. If this claim is true, then it would seem inadvisable to make all of the laboratory specimens of the same consistency, if the results are to be compared with those obtained in practice.

MR. NORMAN G. HOUGH (*by letter*).—In going over the results of the investigation I notice that it has been indisputably shown that when hydrated lime is used in the various mixtures, a greater quantity of water is required to bring the mixture to the same given consistency than that used in the plain mixtures without lime. This additional mixture of water has a value in acting as a so-called internal reservoir which passes off moisture to the cement and is taken up in the process of further hydration, thus causing an increase in strength. **Mr. Hough.**

It is, however, quite unlikely that any increase in strength will be shown in specimens containing lime when stored in a damp closet, which I understand was the method followed by Messrs. Scofield and Stinchfield, as specimens so stored are furnished with an abundance of moisture which consequently produces maximum efficiency of the cement so that the hydrated lime is not called upon to perform in the capacity above mentioned. In view of the great amount of concrete that is placed above ground and does not have the advantage of developing strength except through ordinary atmospheric conditions, it would seem that in order to produce results in

Mr. Hough. the laboratory that may be interpreted from the viewpoint of field results, it will be necessary to conduct an investigation along lines resembling actual field conditions.

In connection with the absorption test, also expansion and contraction, it would again seem that a test planned along lines different from those followed by the authors would be advisable in order to link together laboratory results with the results that are actually being gained in the field, because many stretches of concrete road indicate different results are to be attained in field practice. It would be quite natural in testing for absorption to dry the specimens under heat action to constant weight, but in so doing the specimens containing hydrated lime are acted upon in a manner which will show the specimens to have high absorption. It must be remembered, however, that concrete laid in actual field practice is not subjected to the same treatment under heat action, consequently hydrated lime in field practice preserves the property that reduces absorption from that which is noted when the specimens are completely dried out by oven treatment.

The reason for this difference is unquestionably in the drying process. In the investigation under discussion the moisture has been entirely driven from the hydrated lime by subjecting it to higher temperatures than is ever met with in field practice. Under this treatment the hydrate shrinks and becomes hard and loses its power of readily taking up moisture. On the other hand, hydrated lime that has been brought to a paste by mixing with water, upon drying out in air will shrink and throw off its moisture, but it readily swells and comes back to paste form upon coming in contact with water. The result in the first case, when specimens have been completely dried out by heat action and then immersed, is that the hydrated lime is unable to absorb any moisture, consequently cannot swell to the point of filling the voids, and one would naturally expect higher absorption than without lime.

In the second case, or that of subjecting specimens to conditions such as met with in ordinary practice, it is questionable just how much of the moisture held by the hydrated lime is absorbed by the atmosphere, and at what period the lime would most readily give up its moisture, and whether the moisture, upon eventually leaving the lime, goes into the atmosphere or whether it is taken in by the cement in the process of further hydration, as mentioned earlier in this discussion. It would seem that in all probability in actual practice the lime holds the moisture, each particle remaining in a swollen condition to fill the void, and in this case the absorption would be lower than the results as found in the investigation under discussion.

The value of any material in building construction is determined by the results it produces in the field, and the rapidly increasing use of hydrated lime in concrete construction by prominent engineers for the purpose of improving the quality of the hardened structure, must be accepted as evidence that hydrated lime will prevent segregation by permitting the concrete to be placed under practical field conditions without the use of excess water.

Mr. Anderson.

MR. W. P. ANDERSON.*—My company, the Ferro Concrete Construction Co., has used both hydrated lime and lime slaked ourselves, and our superintendents prefer, of the two, to use hydrated lime. They found the hydrated lime handier on the job than any other kind. The question that naturally came up to us first was whether it would pay. When you add hydrated lime to concrete you are increasing the cost of the aggregate, but we have found that that increased cost is more than made up in the labor saving. We can use chutes where a percentage of about 5 per cent of hydrated lime is used, where we cannot use the chutes otherwise. We have had chutes that have clogged up and caused a great deal of annoyance and trouble by our constantly having to clean them, and by the addition of lime we have been able to use the chutes under the same condition of slope with the hydrated lime and have had no trouble at all.

I believe that the resultant concrete looks better, that you do not have so much patching and pointing up to do afterwards. There is a labor saving at the time of placing and a labor saving afterwards in results. We also found it of advantage for a water-proofing material in so far as its affecting the strength of the concrete goes.

Mr. Ashton.

MR. ERNEST ASHTON.—It is very apparent from the discussion this morning that the actual standing of hydrated lime in the profession of concrete engineering is based primarily upon observation. It is self-evident that there is not sufficient tabulated data to give one a true interpretation of its value from a strength point of view. It is self-evident that the continued increase in strength with time in concrete shows a partially continuous action on the part of the actual material in concrete, namely the cement. In order to continue the cement's activity it is necessary that a certain amount of moisture be present. My idea regarding the use of hydrated lime is this: We know that the water content in hydrated lime is simply held there in a mechanical form; that while the chemical reaction is going on with the cement, requiring a further absorption of water, the chemical activity of the set cement overcomes the mechanical bond between the calcium oxide and the water, in consequence of which you are aiding and increasing the activity and probable hydration of the cement by the absorption of this mechanically held water.

Professor Scofield and Mr. Stinchfield.

PROF. H. H. SCOFIELD AND MR. M. J. STINCHFIELD (*by letter*).—The value of laboratory tests has been established and of course needs no discussion at this time. We have learned by experience that tests made in the laboratory where the variables can both be limited and controlled must and should have their proper significance in the study of any problem. It is to be understood also that the true investigator does not stop with merely experimental results, but should use his utmost energy to analyze these results, bring out the facts and apply them to the problems at hand.

The subject of waterproofing a concrete road is an important one. When once a procedure, such as the admixture of hydrated lime, has been adopted

* The discussions from here on were made by speakers or writers who did not have Prof. Scofield's and Mr. Stinchfield's paper before them.—EDITOR.

P.ofessor Scofield and Mr. Stinchfield.

to accomplish this purpose, it is a legitimate undertaking to inquire into the effect of this addition on all of the qualities of the concrete as well as that of resistance to penetration of water. Carefully planned laboratory tests should have their proper significance in such an investigation.

Storage of Test Specimens.—These were stored in damp air to give uniform conditions as to moisture and also as a median line of treatment fair to both mixtures. A dry storage would be manifestly unfair to the no-lime concrete. The authors have made some tests to show that concrete allowed to dry out in storage sometimes attains little more than one-third the strength of the same concrete when stored in damp air. On the other hand, concrete that has been stored in water has a considerable higher strength, especially after long time immersion, than concrete stored in damp air. It seemed, therefore, that damp air storage gave a condition that neither enfloated or detracted from the strength of either mixture. As a practical commercial proposition, it would seem that the retention of water in the concrete should be attained by the cheaper method, whether by admixture of hydrated lime or by adequate artificial protection, during hardening.

Absorption of Water.—The writers are very interested to know the possible explanation as given by Mr. Emley for high absorption and low permeability. It is interesting to note that whichever point of view is adopted, the absorption is high. This fact is of importance in concrete road and other problems.

Amount of Water and Consistency.—The writers are in hearty agreement with Mr. Emley in his remarks concerning consistency and chuting. However, the subject demands more investigation before the facts can be stated.

FRICTION TESTS OF CONCRETE ON VARIOUS SUB-BASES.

By A. T. Goldbeck.*

The rational design of any structure, whether it be a bridge or a pavement, necessitates a knowledge of the stresses to which it will be subjected. In a framed structure, stress analysis is comparatively simple, and may be made with practical surety. When one attempts, however, to analyze a concrete road slab, a bewildering maze of uncertain forces are found, forces that are so problematical that the rational design of concrete pavements is now rendered an impossibility. The uncertainty lies not so much in not knowing what forces exist as in an almost total lack of knowledge regarding their magnitude. We know that a concrete road slab is acted upon by forces which tend to slide it along its supporting base, which bend it, warp it, subject it to direct compression or tension, or to a combination of direct stresses and bending, forces of shear which tend to abrade its surface, and disrupting forces which weaken it locally. Practically none of these forces is known, even approximately, and until they are known, progress in concrete road building must rest with excellent judgment based on precedent. This lack of knowledge is undesirable, and the present series of tests was made to throw light on at least one of the forces acting on concrete roads, namely, the force of friction at the base.

It has been shown by several investigators that concrete elongates and shortens due to two principal phenomena—change in temperature, and change in moisture content. A coefficient of 0.0000055 per degree F. seems to accurately express the effect of temperature change. Tests made by the author† on a number of large specimens of concrete demonstrate that moisture also plays an important part in influencing the length of concrete structures. When the moisture dries out, concrete shrinks, and may shrink from this cause as much as 0.08 per cent. On the other hand, when the moisture is retained, expansion ensues to the extent of 0.01 per cent. Again, if the concrete is subjected to conditions alternately wet and dry, alternate expansion and contraction must inevitably result. Carefully conducted field measurements made with a specially designed instrument corroborate laboratory measurements, and show that concrete pavements are subjected to considerable motion over their sub-base. Of course, heavy masses of concrete cannot be slid over the ground without considerable frictional resistance being offered to their sliding, and various kinds and conditions of sub-bases vary the amount of friction.

In order to study the effect of the movement of pavements on the internal

* Engineer of Tests, U. S. Office of Public Roads and Rural Engineering.

† The Expansion and Contraction of Concrete While Hardening, by A. T. Goldbeck, 1911 Proceedings of American Society for Testing Materials.

The Expansion and Contraction of Concrete and Concrete Roads, by A. T. Goldbeck and F. H. Jackson, Jr., U. S. Office of Public Roads Bulletin now being printed.

stresses developed, let a hypothetical case be considered in which the pavement after being laid is subjected to a rise in temperature and, in addition, due to weather and sub-base conditions, is never allowed to dry out. Such a pavement will necessarily expand and the ends of the slab will move away from the center. Friction will act to resist this movement and in this way becomes an external force producing almost direct compression in the concrete. Should the frictional resistance exceed the compressive strength of the concrete, a compression failure will result. Assume another case in which the temperature decreases considerably after the laying of the pavement, and let the concrete dry out due to warm, dry weather and a well-drained sub-base. This concrete will shrink, and the ends of the slab will approach its center. Friction will act against the movement, and tension will be produced in the concrete. Concrete is weak in tensile resistance, and not much shrinkage is required to crack the pavement if the sub-base is very rough. From these considerations, it will be recognized that as a step toward the rational design of concrete pavements the determination of frictional resistance at the base is necessary, and the following investigation was made with this ideal in view.

DESCRIPTION OF SPECIMENS.

The bases of concrete roads built in the past have been of varying character. Loam, clay, old macadam, gravel, sand and many other kinds of material support the concrete, and some offer more frictional resistance than others. In laying out the present series, therefore, cognizance was taken of this fact and a number of different bases were used.

Shallow ditches 6 in. deep, 3 ft. wide and 7 ft. long were first dug in the soft clay soil of the Arlington Experimental Farm belonging to the U. S. Department of Agriculture. Filling material forming the sub-base was then deposited under the supervision of a trained road engineer, tamped solidly and smoothed, or otherwise treated, in readiness for the placing of the concrete. A 1 : $1\frac{1}{2}$: 3 mixture was used, machine mixed to a medium wet consistency, and the slabs cast were 2 ft. wide by 6 ft. long and 6 in. thick. The various sub-bases prepared were as follows:

1. Clay, smooth top surface.
2. Clay with cobble stones partly rolled in surface.
3. Broken stone, $\frac{3}{4}$-in. to dust, flat top surface.
4. Concrete base, top surface troweled smooth.
5. Loam, smooth top surface.
6. Sand, top surface smoothed and oiled with heavy flux oil.
7. Clay, surface scored to make it uneven.
8. Gravel, $\frac{3}{4}$ to $\frac{1}{4}$-in., flat surface.
9. Broken stone, 3 in.
10. Concrete base, troweled surface, oiled with heavy flux oil.
11. Sand, surface smooth.
12. Clay, oiled with heavy flux oil.

The first set of tests was made when the specimens were one month old. A frame made of a piece of ½-in. round steel bent to surround the specimen was fastened to the pulling chain, which in turn was linked to a spring dynamometer. Force was applied by means of a light steel rail used as a long lever. Two men at the end of the lever were able to apply a constantly increasing force with great steadiness. The movements of the slab were read by means of a Berry strain gage, and in this way the movements corresponding to known loads were obtained.

On Fig. 1 are plotted the forces required to slide the specimens, together with their corresponding movements. Each specimen weighed approximately 870 lb. The ground during these tests was damp but very firm. It will be noted that movement takes place almost as soon as the load is applied in all cases except that in which large broken stone was used in the base. Here there is great friction from the start until finally when a load of 1000 lb. is reached, slipping begins to take place. Great force was required to start the slab after which no more was necessary than in some of the other bases. Apparently there is no such thing as a constant coefficient of friction, but this varies depending on the displacement. When the slab on the loam base had been slid 0.035 in., the load was gradually released, and the return movement noted on the strain gage. This movement is shown by the dotted curve. The sub-base seemed to behave in a somewhat elastic manner as the slab actually recovered considerable of its forward movement. This can be accounted for only by the partial springing of the sub-base back into its original position, carrying the slab along with it.

The values of the frictional resistance offered at various displacements are given in Table 1.

TABLE 1.—FRICTIONAL RESISTANCE OF CONCRETE ON VARIOUS SUB-BASES.

(Bases were somewhat damp but very firm. Weight of specimen, 870 lb.)

Kind of Base.	Movement.	Force.	Coefficient.	Movement.	Force.	Coefficient.	Movement.	Force.	Coefficient.
Level clay	0.001	480	0.55	0.01	1130	1.3	0.05	1800	2.07
Uneven clay	0.001	500	0.57	0.01	1120	1.29	0.05	1800	2.07
Loam	0.001	300	0.34	0.01	1030	1.18	0.05	1800	2.07
Level sand	0.001	600	0.69	0.01	1080	1.24	0.05	1200	1.38
¾-in. gravel	0.001	450	0.52	0.01	960	1.10	0.05	1100	1.26
¾-in. broken stone	0.001	380	0.44	0.01	800	0.92	0.05	950	1.09
3-in. broken stone	0.001	1060	1.84	0.01	1550	1.78	0.05	1900	2.18

The same specimens and a few more that had been completed in the meantime were again pulled after a period of about a month and a half. During this interval the ground was frozen solid almost all of the time, but the day before the tests a thaw set in, and, combined with rain, served to make the ground exceedingly muddy. The bases of most of the specimens were thoroughly saturated with water which stood on the surface in small puddles. The results obtained under such conditions are shown on Fig. 2,

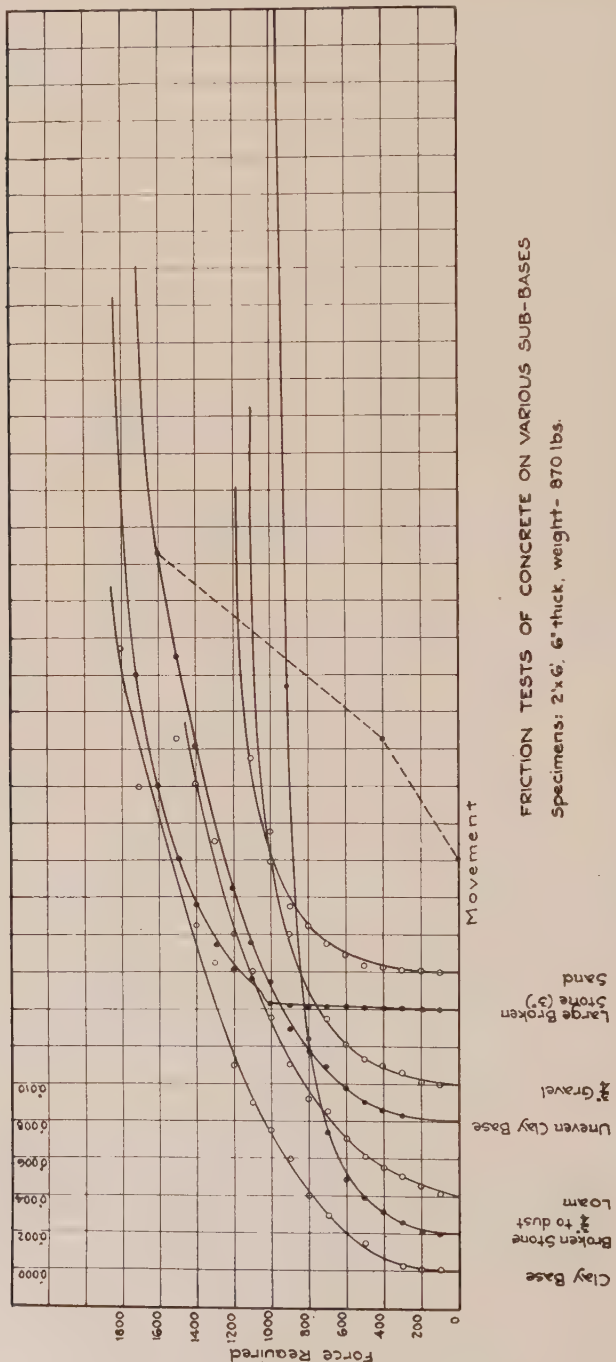

FIG. 1.—CURVES SHOWING MOVEMENT OF TEST SLABS ON DIFFERENT BASES.

Specimens—2 x 6 ft., 6 in. thick; weight, 870 lb.

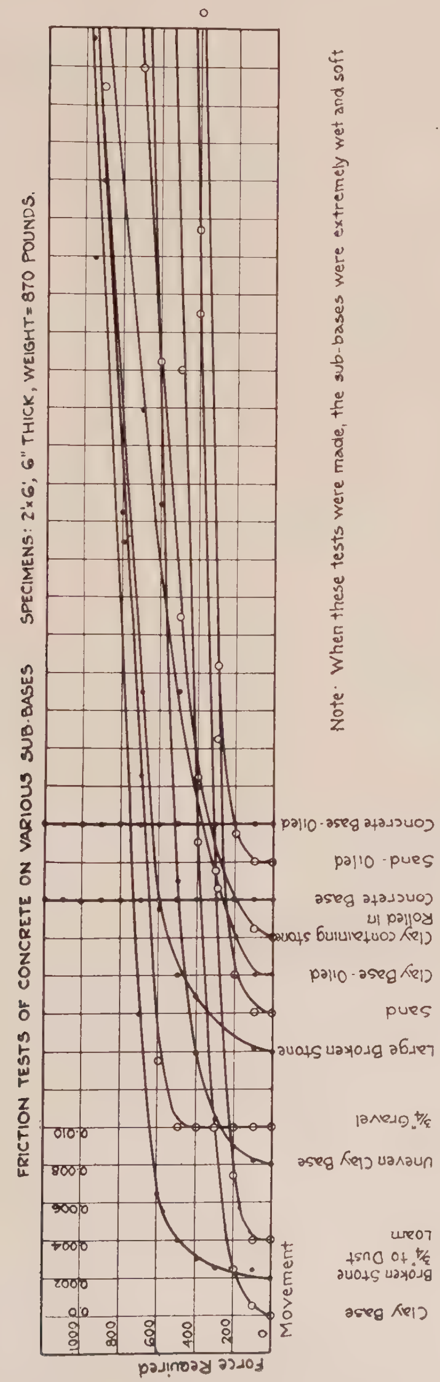

FIG. 2.—CURVES SHOWING MOVEMENT OF TEST SLABS WHEN SUB-BASES WERE EXTREMELY WET AND SOFT.

Specimens—2 x 6 ft., 6 in. thick; weight, 870 lb.

and it will be noted that the friction is considerably lower than it was in the previous tests. The water therefore acted as a lubricant and permitted of the easy slipping of the slabs. The values of the frictional resistance offered at various displacements are expressed in Table 2.

TABLE 2.—FRICTIONAL RESISTANCE OF CONCRETE ON VARIOUS SUB-BASES.

(Sub-bases thoroughly saturated with water and surrounding ground exceedingly soft. Weight of specimen, 870 lb.)

Kind of Base.	Movement.	Force.	Coefficient.	Movement.	Force.	Coefficient.	Movement.	Force.	Coefficient.	Movement.	Force.	Coefficient.
Level clay	0.001	120	0.14	0.01	300	0.35	0.05	500	0.58	1.5	950	1.09
Uneven clay	0.001	200	0.23	0.01	460	0.53	0.05	620	0.71	1.4	925	1.06
Loam	0.001	150	0.17	0.01	260	0.3	0.05	410	0.47	0.75	925	1.06
Level sand	0.001	140	0.16	0.01	280	0.32	0.05	400	0.46	0.75	875	1.00
$\frac{3}{4}$-in. gravel	0.001	510	0.58	0.01	640	0.73	0.05	950	1.1	0.5	1050	1.2
$\frac{3}{4}$-in. broken stone	0.001	400	0.46	0.01	660	0.76	0.05	940	1.08	2.0	1160	1.33
3-in. broken stone	0.001	240	0.28	0.01	630	0.73	0.05	900	1.04	0.875	1625	1.87
Oiled clay	0.001	150	0.17	0.01	410	0.47	0.05	850	0.98	1.25	1425	1.64
Clay and cobble-stones	0.001	140	0.16	0.01	410	0.47	0.05	710	0.82	1.75	1260	1.45
Concrete base	0.000	2500+	2.9+	0.00	2500+	2.9+	0.00	2500+	2.9+	0.00	2500+	2.9+
Sand, oiled	0.001	180	0.21	0.01	280	0.32	0.05	480	0.55	0.375	800	0.92
Concrete, oiled	0.000	2500+	2.9+	0.00	2500+	2.9+	0.00	2500+	2.9+	0.00	2500+	2.9+

The results in Table 2, when compared with those given in Table 1, show very clearly that much depends upon the moisture condition of the sub-base. A wet sub-base permits the concrete to slide very much easier than does a dry sub-base. This apparently also applies to the specimens mounted on broken stone and gravel bases, particularly when the movements are small. In order to try the effectiveness of a heavy oil coating for decreasing the friction at the sub-base, several specimens were made having sand, clay and concrete sub-bases treated with a heavy flux oil.

The oil was applied while hot and was spread over the surfaces with a broom in a layer of sufficient thickness to completely cover the sub-base. The concrete was deposited on these oiled bases immediately. So far as can be noticed, the oil is not effective in decreasing the friction in any of the bases, even when applied to the concrete base that had been carefully troweled smooth. It was impossible to slide the concrete specimen with respect to the concrete base, even when a load of 2500 lb. was applied. No attempt was made to apply a greater load than this, because 2500 lb. was the limit of the dynamometer. When the maximum load had been reached in some of the specimens it was slowly released, and the recovery of the specimen noted. In some instances this amounted to almost $\frac{1}{4}$ in. It thus becomes very plainly evident that when the concrete slides over the sub-base there is a considerable amount of yielding of the base material, and it is therefore to be expected that if the material were fairly homogeneous, and the movement of the concrete took place slowly, the material of the sub-base would gradually yield under this movement, and would thereby offer less resistance than these tests seem to indicate.

In considering friction results, the maximum resistance are naturally of greatest importance, since these create the greatest stresses. Those given in Table 1 are therefore of more interest than those in Table 2, and these results very clearly show that the friction varies considerably in the sub-base, depending upon its character. The indications are that the friction can be greatly decreased if proper care is given to the preparation of the sub-base. Every ridge or depression that is in the sub-base surface before concrete is deposited furnishes an additional grip for the concrete, thereby tending to deform elastically a larger amount of base material, and thus offering greater resistance to the sliding of the concrete.

The formation of transverse cracks in concrete bases can readily be ascribed to direct tension due to frictional resistance at a time when the concrete is contracting, whether this is caused by decrease in temperature, or by drying out of the moisture. The test results show that the coefficient of friction can readily vary from almost 0 to something over 2 or more depending upon the movement of the concrete and the character of the sub-base. The distance between transverse cracks is dependent upon the coefficient of friction, and the total force of friction must extend over this distance.

Calling the coefficient of friction f, the distance between cracks D, the weight of the pavement per sq. ft. w, we may write the equation: $f \times w \times D =$ tensile strength of concrete per foot of width. Assuming the tensile strength of concrete to equal 200 lb. per sq. in., the pavement to be 6 in. in thickness, the weight of a cubic foot of concrete to be 150 lb., and the coefficient of friction to equal 0.5, this equation reduces to: $0.5 \times 75 \times D = 72$ sq. in. $\times 200$, or $D = 370$ ft. If the coefficient of friction equals 2, the distance between cracks equals 90 ft.

It is conceivable that conditions may exist in which the frictional resistance may mount much higher than this, and may be excessive in spots. It would seem desirable to reduce the friction by careful preparation of sub-base so that it is made as smooth as possible. It can very readily be shown mathematically that the forces of friction aid the force of gravity in causing concrete slabs to move down hill. The effect of this movement sometimes shows up very plainly in expansion joints, as it tends to make one side of the joint ride up on the other. The pavement may thus be compared to a train of cars, each slab bumping into its neighbor, and preventing the expansion joints from functioning properly. If the forces of friction are to be controlled and rendered capable of analysis, would it not be well to prevent free movement of the slab at some point, so that the length along which the forces of friction are acting may be definitely determined? Finally, would it not be well to pay more attention to the preparation of a smooth sub-base having minimum friction?

THE CONDITION OF THE WAYNE COUNTY CONCRETE ROADS.

By A. N. Johnson.*

The concrete roads of Wayne County, Michigan, have attracted more attention and given rise to a greater amount of comment than those of any other locality. While these concrete roads are not the oldest to be found in service today, they do, however, constitute the first extensive county system of concrete roads to be built, and as a consequence have been the indirect cause of the construction of many hundreds of miles of concrete highways in all parts of the United States.

Wayne County leads in the number of miles of concrete roads actually constructed in one county; there being today—January, 1917—150 miles of concrete roads in service. The total mileage of the Wayne County concrete highway system, when completed, will be approximately 350 miles.

Visitors to the number of many thousands from every section of the country have inspected these roads. During good weather, scarcely a day passes that does not see one or more delegations of roads officials on an inspection trip over these justly famous roads. Visitors come from Texas, Mississippi, Maine, New York, Oregon and California; and all who have seen these concrete roads of Wayne County have realized the wonderful service they give and their immense value towards the development of the county.

As many counties in various parts of the country have begun or contemplate the construction of a system of concrete highways, a critical statement of the actual condition of the Wayne County concrete roads should be of value. Such a statement will be here undertaken.

It should first be stated frankly that the writer approaches this subject fully convinced of the practical value of concrete highways. This conviction is based on experience in their actual construction as well as a country-wide observation of this type of roads in service under a broad range of conditions.

The facts and impressions upon which this account of the concrete roads of Wayne County is based, are derived from many careful personal inspections—the last one being during the summer of 1916—as well as reports of engineers working under the direction of the writer; which, in addition to other notes, comprise a very complete series of photographs taken during November, 1916. Advantage has also been taken of the reports of Professor Cox of the University of Michigan; who has had these roads under close observation for a number of years, and has made available to the writer some very valuable data.

What is a Defect in a Road.

Before presenting any description or drawing any conclusion, it is necessary that a clear understanding shall be kept in mind as to what constitutes

* Consulting Highway Engineer, Portland Cement Association.

a defect. In dealing with highways, a defect is that which impairs the service the pavement is to render. The question to be considered is, with a particular construction, do the various changes which such construction undergoes constitute a hindrance to public travel? Do such changes give evidence of a development which will in time become the source of inconvenience to traffic? If so, they constitute defects. As a corrollary the form of construction that endures without the development of changes that cause inconvenience to traffic, is a success. Thus, if an uneven surface develops, which increases the tractive resistance; or if a wavy or rutted surface, or holes, develop—all will agree that travel is made more uncomfortable and the pavement has developed defects. But, if a joint that has been constructed in the pavement, or a crack has developed, over which traffic passes without the slightest hindrance or inconvenience, it is submitted that such are not defects. And if, as ample experience has shown, both cracks and joints can be effectively maintained at very small expense continually in a condition which offers no impediment to traffic, they are not to be regarded, nor can be considered, as constituting even a potential defect. Many adverse criticisms of concrete roads are based on the fact that they are cracked; a slab that is cracked is reported as defective. The total number of such cracked slabs is recorded, and the statement made, for instance, that a certain percentage are defective. If the concrete slab composing a highway surface is to be regarded as rendering a similar service to that of a lamp chimney, then a crack is a defect. But the service which a concrete highway renders is in no way impaired by a crack. Were they not made so plainly visible by the band of black tar with which cracks are filled—if properly maintained—they would give occasion for no comment.

Should the number of cracks that occur in a given slab become so numerous as to divide the slab into comparatively small pieces, there undoubtedly would be occasion for some anxiety as to whether these small pieces into which the slab is thus divided will maintain themselves without displacement one with another. As a matter of fact, an occurrence of this kind is very rare and is indicative of faulty construction as to the sub-grade, drainage and the character of the concrete. On the Wayne County roads there is to be observed no slab in which cracks have developed in such numbers as to give rise to any apprehension on this account.

Before commenting in detail upon the condition of the Wayne County concrete roads, the following statement, made by Mr. Edward N. Hines, chairman of the Board of County Road Commissioners of Wayne County, in a paper read by him before the Portland Cement Association in Detroit, September 13, 1916, is significant:

> "About four years ago, I read a paper before this Association on the subject of Wayne County's concrete roads. At that time I made the following statement:
>
> "'These (concrete) roads are starting in their fourth year and, barring a few cracks, are as good as the day they were built. Practically nothing has been spent on their surface for maintenance. On the basis

of three years' thorough trial, I stand committed to the use of concrete for country roads. I believe concrete to be an ideal form of paving for village, city residence streets and alleys. This is not a statement born of enthusiasm, nor made on the spur of the moment, but a cold-blooded, dollars-and-cents view, based on results obtained after careful consideration of all the facts available and all experience undergone.'

"These roads are now in their eighth year. Time has substantiated the statement which I made four years ago, so I now reiterate it."

The general traffic of Wayne County is heavy. Obviously the roads immediately adjacent to the city of Detroit carry the heaviest traffic. A careful traffic census of some of Wayne County's road was taken in 1912. The enormous increase of traffic renders this census of little value in estimating the present conditions. In general, the travel from Detroit northerly consists of automobiles and heavy motor-driven trucks, while the travel westerly and southerly combines considerable steel-tired traffic with motor-driven vehicles. A conservative estimate of the traffic on the improved roads within a few miles of Detroit would indicate a combined total of 1500 to 3500 vehicles daily, these figures being 50 to 100 per cent above the actual count of the census taken in 1912. Woodward Avenue, the oldest concrete road, now in its eighth year of service, and also the heaviest traveled of Wayne County's roads, has carried an estimated total traffic of 7,000,000 vehicles from motor cars to heavy traction engines.

LATER ROADS IN GOOD CONDITION.

The roads built by Wayne County in the past two or three years call for no detailed description. No surface wear on these roads is apparent. Occasionally there will be found a small pit-hole or pocket, due to some minor fault in a particular batch of concrete, which may have contained a few pieces of foreign material, such as a lump of clay, piece of wood or coal. These small pockets or holes are readily maintained by filling with a small amount of tar and covering with sand. A few cracks have developed; but none of them is serious. The Tenth Annual Report of the Wayne County Commissioners, on page 37, remarks that "Although cracks are no asset to a concrete road, we have proof that, properly cared for, they are no detriment. The only objection that can be raised to them is that our methods of repair necessarily draw attention to the location of the crack, because of the black streak left by the tar after the required maintenance has been done. This objection, however, does not come from our road users or property owners by whose frontages these roads pass; but by engineers, road builders or salesmen with something to sell."

It should also be remarked that in 1911 the specifications under which the Wayne County roads were laid were substantially changed. Originally the proportions used for the concrete were 1 : 2 : 4, which proportion in subsequent work was changed to 1 : 1½ : 3. The shape of the sub-grade was changed from a convex to a flat surface, the crown in the pavement being obtained by additional thickness of concrete at the center. Also more care-

ful selection of fine and coarse aggregate was made, special effort being made to secure only washed material. All of these features, combined with greater experience in carrying out the details, have resulted in a much higher grade of work than had been the case with the earlier roads. While the general condition of the old roads is good today, there is every reason to expect that when the more recent work reaches a like age, they will be in even better condition.

THREE SHORT STRETCHES DEFECTIVE.

Further discussion of the detailed condition of these roads will be confined to those built in 1909 to 1911 inclusive. There are three pieces of road, totaling a little less than three miles which have become defective and are failures. Naturally, those interested, for some selfish reason, in giving an impression that the concrete roads of Wayne County are going to pieces, base their allegations upon the condition of these three poor sections; although when questioned closely, they will admit no other section of these roads is at present in bad condition, it is implied that they shortly will become so. The facts are that on all three poor sections a defective condition developed within a few months after the roads were opened. If all the other roads constituting the Wayne County system were of the same quality, they should necessarily show the same development of defects. But no such thing has occurred. The defective sections include one-half mile on the Fort Road, built in 1911; one and three-quarter miles on the Gratiot Road, built in 1910; and six hundred feet on the Grand River Road, built in 1910.

The half-mile stretch on the Fort Road was built by contract, and the resulting work was very poor, so much so that the County Board refused acceptance. They finally compromised with the contractor, allowing him to place a covering of tar over the work at his expense, and accepted the job. It has never been satisfactory.

The one and three-quarter miles on the Gratiot Road was built by day labor, using unwashed materials. Traffic was allowed upon the road very soon after the concrete was placed. It was noticed within two or three months thereafter that the surface of the road was showing an undue amount of wear. Shallow holes were soon found and the surface became very uneven. This road has been covered with a layer of tar and sand similar to the treatment that was applied to the Fort Road. On both of these roads this surfacing has been worn into ridges, so that neither road is today a comfortable road for traffic.

The 600 ft. on the Grand River Road was constructed in similar manner to the rest of the work done in 1910 on this road, but it was noticed on this particular portion of the work that some difficulty occurred. It was remarked at the time by those in charge of the work that they did not expect this section to prove satisfactory—and their prophecy proved to be true. The whole surface soon roughened somewhat similarly to what happened on the Gratiot Road, some places wearing deeper than others until great inconvenience was caused to traffic. In repairing this section the Commissioners were desirous of making an experiment of covering the surface with a thin layer of concrete.

As the 600 ft. would be rather a short piece, they covered a section approximately 3000 ft. long, which extended from one cross-road to the next cross-road; as it would be necessary in any event to close this section of the road to traffic during construction. The additional 2400 ft. were in good condition and were covered merely to make the experiment of sufficient magnitude to make it a real demonstration. The results of this experiment are under observation. This is the history of the three sections which have failed and are the only sections out of the 150 miles constructed which have shown or developed any such defective condition.

How the people of Wayne County regard their concrete roads is best answered by the fact that Wayne County has adopted concrete as the standard form of road construction and that practically no other type of road has been built during the past seven years. The opening of a new concrete road signalizes a wonderful increase of traffic. Not only do the people living along an improved road use it more, but many come miles from all directions to use the concrete in preference to their accustomed route. It is safe to state that traffic is increased three-fold.

The farmers of Wayne County at first opposed the concrete road, and because of this opposition about 22 miles of gravel road were constructed. These same farmers are now urging the Road Commissioners to rebuild these roads with concrete, which will be done in the near future because of the excessive cost of maintaining the gravel roads in usable condition.

During the past few years the adjacent counties of Monroe, Washtenaw and Oakland have built and are building a large mileage of concrete roads; as are the smaller neighboring cities and villages. Considerable sums of money have been paid as voluntary contributions by Wayne County residents fcr the improvement of roads in adjacent territory, with the proviso that these roads be built of concrete.

MAINTENANCE OF CONCRETE ROADS IN CONNECTICUT.

By W. Leroy Ulrich.*

The State of Connecticut did not take up the construction of concrete roads as early as many of the states in the middle west. The construction of the first concrete road in this state was commenced in the summer of the year 1913. It was built to replace a worn-out stretch of macadam and the type was adopted for the following reasons: First, the amount of motor vehicle traffic was increasing as well as the carrying capacity of motor trucks. How far this would increase could only be inferred. Second, the increasing cost of the maintenance of ordinary waterbound macadam caused by these conditions was fast reaching a sum so great that sufficient money could not be obtained to properly take care of its maintenance. Third, the commissioner realized that to build roads which were not strong enough to withstand the future traffic was false economy. Fourth, the original cost of the type to be chosen could not be so high as to prevent a considerable mileage being built to replace the poorer types by hampering the construction of new work. Fifth, the maintenance cost of the type chosen must be low in comparison to the original cost.

After studying the question thoroughly, it was decided to try a concrete road because its first cost was moderately low, in comparison to the small cost of maintenance which was anticipated. Since that time there has been constructed by the state, 23.89 miles in 1914, 8.73 miles in 1915, 14.40 miles in 1916, making a total of 50.53 miles in the care of the Repair Department at the present time. This type of road cost from $1.10 to $1.35 per sq. yd in 1914, and the last contracts let in 1916 cost the state about $1.80. Estimates for the coming year are based on an anticipated cost of about $2.00. At the earlier prices, this type of road was distinguished for its moderate first cost, but with the continued rise in the price of the essential ingredients, it will soon be forced out of the class of moderate cost pavements and into competition with the so-called permanent pavements.

In order to understand the maintenance of any road, it is essential to know the methods which were used in its construction. The State of Connecticut builds its mixed concrete roads on a flat sub-grade, of a 1 : 2 : 4 mixture 6 in. in depth at the edges, and with a cross-crown of ¼-in. to a foot. The mixture is reinforced where necessary, and joints are placed transversely about 30 ft. apart. Ordinary tarred paper joints are used, the number of thickness of paper depending on the temperature at the time the surface is placed. On roads containing car tracks, joints are placed longitudinally about 2 ft. from and parallel to the rail.

Surface finish is obtained by hand floating and brooming. Concrete roads have also been constructed by the grouting processs. The surface

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resulting from this method of construction has thus far met the strain of severe modern traffic very well.

Having obtained a surface which appears to better withstand the wear to which it is subjected, do not think that it may be neglected. Every type of surface requires more or less maintenance. No type of road has yet been developed which is sufficient to withstand the combined effect of frost action, disintegration and the wear resulting from modern traffic, without paying the price of constant supervision and maintenance, be it large or small.

That portion of the Connecticut Highway Department which is responsible for the repairs, maintenance and reconstruction of the highways under its control, has been organized to give as nearly as possible this continuous service which is absolutely necessary. It is in charge of one Superintendent of Repairs, who reports to the Commissioner. The state is divided into ten districts; these being laid out in such a manner that any portion may be easily reached from a central point, at which is established an office with a Supervisor of Repairs in charge. The supervisor reports directly to the Superintendent of Repairs. The assistant superintendent is also in charge of one of these districts. Each of these is in turn divided into sections with a foreman in charge of each section, who reports in turn to the Supervisor of Repairs of the district in which his section lies. These foremen may act alone as a patrolman, or may handle up to ten or fifteen men, depending upon the work necessary in their section and the season of the year. In order that this system be not too rigid, there are in each district one or more floating gangs which may be transferred from place to place to take care of reconstruction, oiling or any work which might be neglected were they not available. This organization handles all the work of the repair department, with the exception of large reconstruction contracts.

The small portion of the total maintenance which is expended upon the concrete roads may perhaps best be described by enumerating the various defects which have made this work necessary.

Certain features of the construction of concrete roads are, in themselves, causes of wear. The most prominent of these are joints. Any break or unevenness in the surface of a concrete road is an invitation to the traffic to start the process of destruction, and most of the work which has been necessary on the surface of the concrete roads of Connecticut has been the treatment of the joints. Patented joints do not seem to correct this evil, as practically as much trouble has been experienced with these as with the common tar paper variety. The state has not experimented with leaving out the joints or lengthening the distance between them, because in its experience with grouted concrete pavements, cracks have developed in them with surprising regularity. A straight well-made joint can be much more easily taken care of than any ragged crack which might result if no joint were used or if the distance between were lengthened. All joints are treated with tar during the hot weather of the first summer after the completion of the work, and as often thereafter as necessary, but only in warm weather. A tar kettle of about 150 gal. capacity, a pouring pot of the type used for pouring bituminous filler for brick pavements, a wire push broom and a couple of laborers, comprise the outfit necessary to properly take care of this work. The joint

is first brushed clean, then filled with tar and covered tightly with sand or stone chips which have previously been distributed along the road. The cost of this work varies with the conditions of the joints and price of labor. Another defect which will sometimes appear is due to foreign material introduced into the mixture during the process of construction, such as bits of wood, pieces of coal, barrel bungs and the like, which crush or work out of the surface and afford a chance for wear. It may be said that this condition should not arise, but these foreign materials are found in the surface once in a while, although no one seems to know how they get there. The small holes thus caused are detected before any serious damage is done and repaired with the same process as the joints.

The use of a poor quality of sand caused some trouble on the earlier of the roads by not offering sufficient resistance to abrasion and a resultant slight wear on the surface. This occurred only on the first two contracts, and was discovered and remedied at once by a light cost of hot tar broomed onto the spots which showed wear. This tar was covered with a dusting of sand. Only a few small spots developed and these have given no further trouble.

The use of too wet a mixture, too much hand floating, or a combination of the two, has caused a slight flaking on the surface of one or two sections. This flaking has apparently not yet affected the ability of the surface to withstand abrasion, and no additional treatment has been used.

These inherent or construction defects can be studied, and are now as far as possible eliminated. Very little trouble has been experienced from these causes on the later contracts. On the other hand, even the most careful study of the process of construction and treatment of the sub-grade conditions does not always prevent the occurrence of troubles due to conditions arising from the action of ground or surface water, changes of temperature and similar causes. Longitudinal cracks are the most frequent and annoying results of forces exerted after the work has been completed. These occurred more frequently on the earlier contracts which were not reinforced and also on those which were so situated that the frost could effect an entrance under the road from the sides, lifting each edge and causing cracks to appear when the surface returned to its normal position. These cracks have been treated in exactly the same manner as the joints. Springs and ground water which had never given trouble on the original road, broke up one slab on the third contract. This is the only renewal which has been necessary.

Practically all of the work on the surface of these roads is performed in the same manner as outlined for tarring joints, and with the use of the same materials. A medium heavy tar which requires heating is used. Any tar which will conform to the following specifications will be found satisfactory:

Specific gravity at 70° F.	1.20 to 1.30
Free carbon	10 to 25 per cent
Distilliation:	
Total to 170°	Not over 5 per cent
Total to 315°	Not over 30 per cent
Specific gravity of entire distillation at 60° F.	1.033 to 1.038
Melting point of residue	115° to 160°

The size of the sand or fine stone used with the tar should depend on the size of the defect being treated and should be clean. If defects are treated as soon as they appear, practically no cover will be used larger than torpedo sand.

By following the principle of "A stitch in time saves nine," the surface of these roads have been maintained cheaply and no general surface application has been necessary on any of the contracts.

Actual costs of the maintenance for the various contracts have been kept for a period of two years, and the results are very satisfactory. Twenty-five miles of road have been used in obtaining the figures given, care being used to include those carrying all classes of traffic from the heaviest to the lightest. The actual cost of all tar work applied to the concrete surface has been $38.63 per mile per year, or about 0.4 ct. per sq. yd. of travel path per year. The cost of shoulder work and drainage has been $28 per mile per year, or 0.3 ct. per sq. yd. per year. The actual maintenance costs for the same period over the entire trunk line system of all classes of surfaces has been $585.18 per mile per year, or 6.25 cts. per sq. yd. of travel path per year. A comparison of these figures shows that the concrete roads have been a good investment.

There are also some concrete roads in the state of Connecticut in addition to those constructed by the state. About thirty miles of grouted concrete laid under the Hassam patents and practically all under a maintenance guarantee, have cost for maintenance, over a period of five years, about 1.2 cts. or 0.25 ct. per sq. yd. per year. The city of Hartford and the town of Greenwich have constructed mixed concrete roads which were originally covered with a light carpet coat of tar. None of these carry any considerable steel-shod traffic, and the maintenance costs, while very small, could not be obtained.

The condition of these roads in Connecticut at the present time would indicate that the traffic has had very little effect upon them, and the maintenance cost should not materially increase for several years.

SOME RECENT DEVELOPMENTS IN THE CONSTRUCTION OF CONCRETE ROADS.

By W. M. Kinney.*

So surprising has been the growth in yardage of concrete pavements constructed in the United States that until within the last year or two little time or thought has been given by the engineer and contractor to the development of new ideas and methods relating to their construction. Those constructing this type of pavement for the first time have been content to follow in the path of the pioneers, but the firm establishment of concrete as a pavement of the highest order is developing an ambition to better previous work and expedite its accomplishment.

This condition has been further aided to a large extent by the fact that the construction of concrete roads and pavements is being carried on in larger units than heretofore. Building roads in one-mile sections and letting street and alley contracts in limited yardage does not tend to encourage the purchase or development of labor saving devices nor does it warrant the engineer to request the contractors assistance in experiments to improve quality.

Small jobs attract the contractor with little equipment and the size of the contract does not justify additional purchases. Nowadays, public sentiment, demanding immediate and extensive road improvement, has caused the approval of bond issues and greater tax levies. These have in turn permitted engineers and public officials to award contracts of sufficient size to result in greater economy and at the same time encourage new and useful improvements in the art of road construction. In no division of the work has the latitude thus allowed been more keenly appreciated than in that involving the handling and hauling of materials.

Hauling Material.

Large quantities of heavy materials are required for concrete road construction. These materials must often be hauled great distances over roads which are seldom adapted to heavy traffic. Although the original cost of an industrial railroad to handle these materials makes the expense prohibitive on a small job, as the yardage of the work and length of haul increases, this disadvantage is gradually overcome and the economy of such equipment soon appears. Many influences affect the choice of hauling equipment. Where teams are cheap, they will usually be found the most adaptable, if the haul is short. For longer hauls and where the roads will stand up under such traffic the motor truck finds most effective use. Similarly the tractor and train have their legitimate and effective field. Each of these three modes of transportation for road materials, however, has the disadvantage of destroying to

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a more or less extent the roads over which they travel and of cutting up the sub-grade over which they must carry their load. The latter item is most important and vitally affects the success as well as the efficiency of the work.

As the rolling of the sub-grade is usually done before the materials are hauled, the ruts made by wagons or trucks are seldom filled and compacted as they should be before depositing the concrete. Particularly is this noticeable in wet weather. The use of an industrial railway with cars of size suitable to carry materials required for one batch and depositing direct into a side-loading mixer has much to commend it. The roads over which it travels are not rendered impassable, the sub-grade is not cut up and therefore made unsuitable for depositing concrete; the materials are kept clean and free from dirt and clay often shoveled from the sub-grade, and there is no loss of concrete materials.

Some may argue, and correctly, that on several jobs where this method of transporting materials has been used the costs have been high and the method unsatisfactory. This has very probably been due to one of two causes; either the size of job did not justify such equipment, or else the crew was not properly organized. Mixer output and therefore the amount of road placed are dependent on the regularity of supply of materials. Any breakdown in the hauling equipment means a large loss in time. There must consequently be an extreme nicety of balance between the crew supplying materials and the crew mixing and placing concrete in order to insure uninterrupted operation. That this method has failed on some jobs is not a condemnation of the method but rather of the organization which experienced the failure. Further use and study of this method is warranted both from the standpoint of economy and of equality.

STORAGE OF MATERIAL.

On jobs where materials are placed on the sub-grade much trouble can be saved and considerable economy effected by dumping them systematically along the work so that there will be a minimum trip for the wheelbarrows, and at the same time no overages which must be hauled back or used in the shoulders. To the storage of cement the same applies. A uniform supply of cement at measured intervals along the road gives an accurate and easy check on the amount used. Such storage units are also always ready to withstand a sudden rain storm, thus permitting the use of all men available to get the unfinished concrete under cover.

When a sudden rainstorm comes up the wise contractor will be provided against this emergency by having ready for use a canvas cover and preferably some type of framework on which to support it. This prevents the pitting of the surface by rainfall and the framework avoids unsightly marks on the pavement surface which would be caused by placing the canvas directly on the concrete. Such frames if made of light material can be moved along as the work progresses and will be useful in preventing too rapid drying out of the surface immediately after it is finished. On one job where considerable rain was experienced, the frame was so constructed and the canvas of such width that the workmen could finish under the canvas during even a protracted rain.

MEASURING AMOUNT OF WATER.

We are not sure of the cause of certain defects that appear in concrete roads and pavements but it is certain that the use of an excess of water is responsible for by far the greatest number. Laboratory experiments have demonstrated that the strength of concrete is greatly reduced by the addition of water beyond that necessary to make a workable mix; in fact it is stated on good authority that changes in amount of water have as great an influence on the strength as changes in amount of cement. This suggests then the probability that future specifications will state not only the proportions of cement and aggregates to be used but the amount of water. Automatic devices for measuring or weighing the amount of water for each batch are already in common use.

Closely associated with the amount of water used and quite as important in its effect on strength is the time of mixing. Proper mixing of the ingredients of concrete cannot be effected in less than one minute, and a longer time will usually be found well worth the extra expense. The time of mixing also has an important influence on the amount of water used as it is possible with continued mixing to increase the plasticity with less segregation of coarse aggregate from the mortar.

Several devices are on the market with which it is possible to insure a minute mix. One of the simplest is an egg timer of the hour glass type from which enough sand is taken so that the balance will run through in a minute's time. This can be hung at a conspicuous place on the mixer and is reversed as each charge is made and can be seen by the inspector from a considerable distance. Other styles depend for their operation on time or number of revolutions. One type makes it impossible to discharge the concrete until the full period has elapsed.

FINISHING DETAILS.

Probably no new method used in constructing roads has attracted more interest among engineers and contractors than the use of belts for finishing. Wherever the belt was given a thorough trial the tedious process of finishing with hand floats from a cumbersome bridge has been supplanted. A surface more free from irregulatities is obtained at a considerable saving in cost.

Being a new method, standard practice in the use of belts for finishing has not yet been established but general use indicates a preference for three to four-ply belts, 8 to 10 in. wide. The 8-in. belt is used immediately after the concrete has been struck off, and the wider belt for final finishing. This final finishing should be done as late as possible and still produce a surface of uniform texture. If final finishing is done too soon, hair cracks will result, particularly if a hot, dry wind blows over the concrete. There is also more tendency to draw excess water and cement to the surface, thus producing a pavement which will wear quickly until the coarse aggregate is reached.

Canvas-faced and rubber-faced belts have been tried as well as strips of ordinary tarpaulin. Because of its greater pliability, limited experience seems to favor the rubber-faced belt. For the same reason canvas belts which have been in service are to be preferred to new ones.

Some difficulty has been experienced in the use of belts where stone screenings were used for the fine aggregate. Spots will frequently be left in the surface which must be hand floated in order to give the proper wearing surface. The belt will probably be found advantageous in all forms of road construction but its first use should be watched carefully to make sure that an absolutely uniform texture is obtained.

As the use of a belt produces no appreciable compacting of the surface, a strikeboard with which it is possible to do some tamping of the concrete is recommended. This strikeboard should have a face about 8 in. wide and be provided with long handles similar to those used on a hand plow, as the strain on workmen using such a strikeboard is too great if they have to bend over to do the work. The use of a tamping strikeboard also permits mixing the concrete to a dryer consistency because the tamping brings more mortar to the surface than the usual seesaw motion of a narrow strike.

Belts are found to be particularly advantageous where the road is constructed without joints, as it is usually necessary where joints are used to finish at the joints with a wooden float and steel trowel.

How to strike and finish concrete on wide streets is a problem which has bothered some contractors and made others hesitate to bid on such work. The construction of a suitable strikeboard and its proper operation at reasonable cost is a difficult problem. During the past year, the Illinois Hydraulic Stone and Construction Co., of Elgin, Ill., constructed on Sheridan Road. Kenilworth, Ill., a very excellent pavement on which the proper contour was obtained by hand work, using "lutes" or short strikes, and working to stakes set at frequent intervals. This does away with the inconvenience of using a long strikeboard and is particularly advantageous when streets have variable crowns to provide for gutter drainage. The results speak well for this method and suggest the training of finishers for this class of work.

MACHINES FOR FINISHING ROADS.

During the past year considerable improvement has been noted in the machines for finishing concrete roads. In Wayne County, Mich., the Baker machine is being used with considerable success. Successful also is the work being done in Oakland County, Mich., by the R. D. Baker Co., of Detroit. The old objection to this machine that too wet a mix was required for smooth operation has been overcome and the finishing machine is destined to play a more prominent part in future concrete road construction. Where no joints are used it has particular advantages as extreme care at the joints is usually needed to secure the best results. It is probable that only by employing a machine of this or some other type such as the Cowell finishing machine, used in California, can we have the stiff mix so essential to the best results.

Ultimately we will undoubtedly consider a consistency such as we use now far too wet for the best results and will find it impossible to finish concrete of the desired consistency without some preliminary tamping or vibrating by machine. For street work a vibrating machine of the type devised and patented by R. C. Stubbs of Dallas, Texas, may be found advantageous. This machine is operated over a flexible board mat placed on the fresh con-

crete and should permit the use of a comparatively dry mix although photographs of work done with the apparatus seem to indicate an excessive amount of mortar on the surface of the work.

For installing unprotected joints the methods used heretofore have not, as a rule, been entirely satisfactory. A plank on edge is usually staked across the pavement, the filler is placed against the plank, and when concrete has been deposited on both sides, the plank is withdrawn. This method of procedure is objectionable in that it seriously disturbs the joint, often raising the filler from the sub-grade one to three inches, which practically destroys its filler from the sub-grade 1 to 3 in., which practically destroys its value. To overcome this difficulty a device has been developed and successfully used on several jobs. It consists of a light steel channel or 2 x 8-in. plank, supported edgewise on the side rails (by means of small brackets or standards) a sufficient distance above the side rails to allow for the crown in the pavement and for the $\frac{1}{2}$-in. joint filler extending above the pavement surface. To this channel, or plank, are bolted steel fingers extending below its lower edge, a distance equal to the thickness of the pavement, plus the crown and the projection of the filler above the pavement. These fingers are made from $\frac{1}{2}$ by $\frac{3}{8}$-in. strap steel and are placed 12 to 18 in. on centers. Eccentrics formed of $\frac{1}{2}$-in. round rods are placed opposite each alternate finger in such a manner as to leave an opening somewhat less than the thickness of the filler when the eccentric is closed between it and the steel finger. These eccentrics extend through both flanges of the channel or through the 2 x 8-in. plank and the upper ends are bent at right angles forming a handle by means of which they are adjusted. The joint filler is placed between the eccentric and the steel fingers. The eccentrics are then closed and the device set in place. When the concrete has been deposited on both sides of the filler the eccentrics are loosened and the device withdrawn, leaving the filler in place undisturbed. The device should be set in place with the side on which the fingers are bolted facing the mixer, so that a backing is provided for the filler until the concrete is placed against it. A filler having considerable rigidity is preferable when using this device.

The curing of concrete pavements is an important matter. Plenty of water should be used and applied as soon as possible after the work is finished. On hot, dry days, sprinking of work placed in the morning can be started in the afternoon and almost invariably on all work, sprinkling can follow on the next day after placing. Curing by "ponding" or flooding is becoming popular where conditions will permit such procedure. This method of applying water to concrete roads after they are finished, absolutely insures thorough curing and is destined to become more extensively used.

In the development of systematic and more efficient methods in concrete road and street construction the progress made in the last two years is encouraging. More rapid strides will be made during the year 1917 and years to come. Last year the great cry was for labor and this demand promises to become more urgent. Let us use our ingenuity to devise new ways of building concrete roads and streets with less labor at the same time improving their quality.

DISCUSSION.

Mr. Kinney. Mr. W. M. Kinney.—I would like very much to have Mr. Hines describe the use of the Baker machine and also the construction of a 3-in. concrete surfacing to an old concrete road which he did.

Mr. Hines. Mr. Edward N. Hines.—Personally I favor the machine over all other methods of finishing; you get a denser concrete, eliminate the water pockets and the air bubbles, get a more smooth and level surface and can mix your concrete and place it dryer than in any other manner. The basic idea of the machine is Mr. Baker's, who is a Detroit contractor. The machine is now developed to such an extent that we get a even, level surface, with a very dense, hard concrete. I believe if you would take either a section of the belt-finished concrete or a section of hand-floated concrete, and compare them with the machine-finished concrete, that the latter would weigh more and would absorb less moisture. You use a little more material because you make the concrete denser with a machine finish.

There is no saving in the cost of operation. It requires an engineer to run the machine and one other man to assist around the joints and around the machine. The cost is a little greater, including the depreciation of the machine and the interest on your investment and the higher price that you pay a man to run the engine on a machine. The hand finishers that we formally used were principally Italians whom we trained ourselves and to whom we paid \$3.50 to \$4.00 a day; that is, previous to 1916. Labor conditions are very bad in Detroit; in 1916 we paid common labor \$3.25 to \$3.50 a day, and other labor in proportion.

One crew used the belt finish during the entire summer and built with the belt finish about 9¾ miles. We are using three-ply elevator belting, with a 12-in. belt following after the strike-board and an 8-in. belt as a finisher. It makes a beautifully surfaced road, just as smooth and level as you could imagine and very comfortable to ride over. My objection to the belt finish is the apparent necessity for making the mix very wet.

Regarding the resurfacing job: About three years ago we laid three or four sections 2 in. below grade on one of our roads and then went back after set had taken place and put on 2 in. of new concrete, with the idea of seeing what might happen. Those four sections have been there three years now and nothing has happened, but our thought was that eventually something would have to be done to take care of the wear that would occur in time. I do not quite agree with those advocates of concrete road construction who contend that they are a wholly permanent proposition. They are permanent relatively to some other types; that is all. Our idea was to place ourselves in a position so that when the time came that it was necessary to do any amount of resurfacing, we could resurface with new concrete over an old concrete road at a lower cost than we could put any other material on top of it.

We took a section of Grand River road because we desired to get rid of some stretches there that were not good and from the further fact that this road bears a very heavy mixed traffic. We wanted to see how it would work out with a combination of steel tires, motor trucks and pleasure cars. The length is a little over 3000 ft. **Mr. Hines.**

The road, built in 1910, was rounded off with a 3-in. radius. In the reconstruction we decided to widen from 16 to 20 ft. This we did by breaking off the rounded edge leaving a width of 15 ft., taking off 6 in. on each side with sledges, and then adding the necessary 30 in. of sub-base on each side. Then the old concrete sub-base was sprinkled and immediately on top of the entire sub-base was spread in a thin film a mixture of tarvia heated to about 225° F. On that we placed a $1 : 1\frac{1}{2} : 2\frac{1}{2}$ concrete, using Marquette trap rock reinforced with No. 26 triangular mesh wire. The surface was finished with a Baker machine. The new joints were made to coincide with the old joints, from which the tarvia had been dug out in the earlier reconstruction operations.

The road was opened the earlier part of June and when we went into winter it was in most excellent condition. There had no cracks developed except the few hair cracks which developed right away. The cost was high, because of labor conditions, the time of the year and the fact that we were trying to demonstrate a principle rather than to get the costs down. I believe that under present conditions we could reconstruct it over again for something like 80 to 90 cts. per sq. yd.

The idea of the tarvia was to provide for a little different coefficient of expansion between the two mixes and the two classes of material. We tried out some slabs the preceding year in various ways and when we attempted to break the slabs this tarvia coating did not split off but broke clear through and we thought perhaps there might be some merit in the method.

ARTISTIC STUCCO.

By John B. Orr.*

What great possibilities can be conjured up in these two words. Stucco is among the oldest in some form or other of man's early attempt at the artistic. With all the possibilities and, despite the fact, that there can be found to this day portions of stucco in a good state of preservation after standing the wear of many centuries, there is no other form of building material that has fallen more into disrepute than stucco. This is especially so in the United States. The causes can be largely traced to the slipshod methods of procedure that has gradually crept into our building industry. Today the main point of view or achievement that is looked for is whether a contractor can complete in sixty days what should take three or four times longer. Short cuts are taken wherever they can; things that appear small in the successful completion of the work are sacrificed for time. The boy learning the business does not learn how good to do it but how fast to do it. The view he sees as a successful craftsman is not to do better and try to improve on the specifications for the work but just how much he can scamp and get away with.

Some contractors govern their cost by these methods and we get the results so often noticeable in modern construction, competition in price instead of competition in value or good work. The good contractor who tries to figure at a price that will permit good work, in many cases is forced out of business leaving the field open to the cheaper man and cheaper methods. The old school of craftsman had a different view; they tried to make their work masterpieces just as much so as the artist did his on canvas. They wanted to look at it years afterward and be able to say "I did that," or "I worked on that," and feel the pride that comes from viewing a masterpiece. I believe every form of encouragement and instruction should be given the craft to encourage good work. Your body which sits in convention is an exemplary method for the betterment of the work.

History of Stucco.

We find that stucco was used in building almost as soon as buildings were found to be necessary. It grew from the crude mud huts to the artistic treatment of exteriors to be found in the old world today. Stucco is an Italian term usually applied in Italy to an exterior plastering although we can trace it further back under a different name. The old Egyptians and the classical Greeks used a form of exterior plastering extensively, however, I have always looked upon Italy as the mother of the plastic art and responsible to a great extent for the artistic effects of exterior plastering generally known in this country as stucco. In Great Britain stucco is a somewhat

* Contractor, Miami, Fla.

indefinite term for various plastic mixtures. The great Robert Adam is due credit for the advancement of exterior stucco in Great Britain. He adopted it as a covering over houses built of brick and cobblestone and it was used extensively during his period.

In going over several books in my collection I find that the Temple of Apollo at Delphos and even the first Parthenon under ægis of Pallas was plastered with stucco. Vitruvius calls the exterior plastering Tectorium Opus. This was composed of three coats of lime and sand and three coats of lime and marble, the united thickness not being more than one inch. The first coat was of common but very old lime and sand (lime that had been "soured" three or more years); when it was nearly dry a second and third coat was applied and left fairly straight. The work was then laid over with another two coats of lime and marble and finished with a coat of fine marble powder; this finish of marble powder being troweled into it before it was dry. The marble mortar was beaten to render it tough and plastic. The successive coats of marble mortar were troweled into each other before they were dry. The tectorium was then painted in brilliant colors while it was still fresh. In certain conditions the surface was then rubbed with wax and pure oil for the purpose of adding to the brilliancy and endurance of the colors.

Slabs of this tectorium have been found and preserved from the ruins of Pompeii and Herculaneum and are in the Museum of Portici. Specimens also from the same place are in the South Kensington Museum, London. It was found that some of this work was colored integrally while in others it was colored by the use of a wash which was applied over the surface while it was still fresh. The early workers in stucco had each their different formulas for treating the stucco to make it weatherproof. Pliny mentions fig juice as being used in exterior plaster; elm bark and hot barley water were mixed with the stucco used on Justinian's Church of the Baptist, Constantinople.

Bullocks' blood was employed for this purpose in the mortar for Rochester Cathedral, England. White of eggs and strong mort of malt was used in the lime for Queen Eleanor's Cross, Charing Cross, London, in the year 1300. It is a historic fact that during the building of the Duke of Devonshire's house at Chiswick, the exterior of which was plastered with stucco, the surrounding district was impoverished for eggs and buttermilk to mix with the stucco. My mention of these different methods and treatments is to show the care and wide range of methods and mixtures that was used in the endeavor to make the stucco weatherproof, and the difficulty that the old craftsman had to contend with in getting these results.

Modern manufacture has overcome this to a large extent and has made the path of the stucco workers easier. It is a curious fact that the fountain of possibilities in modern stucco has hardly been tapped. I give for the reasons: first, fear of the permanency of the material; second, neglect by the architects in not studying the possibilities; third, the difficulties in getting the work executed, owing to the ignorance of the craftsman in this branch of the plastic art. In reply to the first, anyone who has traveled or has gone into stucco historically can prove the permanency of the material

before and after the introduction of portland cement as the material. By the introduction of portland cement and waterproofing compound much has been done to simplify and make permanent the mixture. The danger in most cases to be overcome is the manipulation of same.

How to Eliminate Crazing.

My greatest obstacle to overcome has been crazing or check cracking. This I have cured by what I believe to be the only sure method. The richer you get the mix, the more danger there is in check cracking. Rapid drying, heat in cement, soft sand, these all help to cause check cracks. I have taken precautions against these dangers and have done what I could with the local materials that are obtainable here. I had good results in some cases and in some others check cracks did appear despite the fact that I had made every effort to avoid them; I never yet had any to scale or fall off. My next attempt I made using an overwash of liquid stucco. This last method has proved

FIG. 1.—FLORIDA RESIDENCE WITH STUCCO FINISH.

very satisfactory and I have jobs that are two years old and on which there has been no appearance of crazing. In Florida we have several obstacles to overcome although we do not have the freezing weather. We get a very poor sand, that is impregnated to a certain extent with salt. The sand is not sharp enough. We have quick drying weather and strong sun heat, and I believe this to have been a severe enough test to show that the stuff would not craze. To the architect and designer, as a layman, I offer a few suggestions and criticisms. As a general rule they do not give enough study to the possibilities in color effect such as are to be seen in Europe, Cuba and other Latin countries. Then, in ornamentation they seem to forget that they are working in a very plastic material that lends itself to the fullest extent in obtaining lights and shadows.

I believe that to get the full effects, relief work in stucco should have the appearance of being modeled in place with this material. It should not have hard lines and in no case should it have the appearance of carving as in stone. The work should retain all the touches of the modeling, these touches that give the sketchy effect which is lost in the carving in stone.

In the preparing of the models the modeler should accentuate the detail and not attempt to smooth up the model. These markings, when brought out, all serve to make the work plastic and alive. It also helps in obtaining light and shade when colors are used as the finish. Even when the work is colored integrally these markings of the tool all stand out and bring out the work better to the eye when the buildings weather. In other words, he should not attempt to get in the clay any smoother work than he could get if he was modeling with stucco right in place instead of modeling in clay.

Low Relief Work Easily Obtained.

By proper manipulation of colors and attention to the above details great beauty can be obtained from work in low relief. Several jobs which I have under way at present I am using this method on and am getting what I believe good results. I am not attempting to confine the relief work to panels but am using the walls as the background, getting an effect as if the work was actually modeled in stucco and keeping the relief work very low and plastic. These are the touches that give the sketchy effect that is lost in the carving of stone. As a general rule it seems to be the practice of designers in stucco to copy stone; this, in my opinion, is entirely wrong. Stucco is a distinctive material and should be used as such. In my ornamental molding and relief work I use a combination of several colors (which match with the general color scheme of the exterior of the house) to bring out the effect and give light and shade. I use the darker tints in the background and work out the lighter tints to the high lights, blending all the tints by rubbing the one color into the other. By doing this you bring out all the plastic beauty of the modeling and give an artistic appearance to the whole scheme. My colors on stucco I bring out by the use of a wash of liquid stucco. I am quite enthusiastic on this color work and I think it wonderful the effects that can be obtained with its intelligent use.

A study in colors for the stucco of buildings is the work of an artist and should be given this care with due consideration to the surroundings in which the house is to be built. I possibly might be treading on the toes of manufacturers of cement paints when I say that I only use this method when actually called for in some public buildings where the effect is to get something that will always look new and clean; in some cases, of course, this is very necessary but for residential work we cannot do better than try to copy the early Italian stucco effect, this is, to get the results like stucco and not paint. There is something about the technical paints that give to the stucco an artificial appearance. I never like to see a residence that looks like it had come out of a machine-made mold. I look to see the sketchy effect and also like to see the building weather properly, not stay one solid color, but get the soft effect that only a stucco can take on; a blend of several shades which come by age and this is my objection to cement paint on residences; it looks artificial. Its use, in my opinion, as before stated, is limited to certain types of public buildings where the surrounding buildings, street and sidewalk have the tendency to harden the effect.

HOW TO GET COLOR EFFECTS.

Conditions like this call for an entirely different treatment than the residence that sits in grounds where one gets the benefits of the color effect of flowers and foliage. My idea in getting effect and tone to a residence is that a study of the whole scheme, including the landscape work, should be taken into consideration and let the residence become a part of the landscape on which it sits and not make it look like an obstacle that has been put in the way of the beauty of nature. In public buildings there is a big field for the stucco worker in producing the effects that is obtained by the use of terra cotta. Stucco can be made a formidable competitor of this material. It can be made permanent and has as wide a range of colors as polychrome terra cotta. When this is the result that is required, this is the method I use and into which I use cement manufactured paints as a background applied over stucco surface for the color effects. I apply the stucco according to the methods and specifications given later. When the stucco is thoroughly dry I then apply a priming coat of a good cement paint, using the material thin and working it into the stucco surface with the brush, being careful not to use the material too thick so that it will not spoil the texture of the surface, the texture of which should be a smooth sand finish. If the effect wanted is in a blend of several colors my system is to cover the surface of the stucco with two coats of cement paint as mentioned above.

I then mix up my blending materials in the form of a stain, using good mineral colors ground in oil which I thin down with prepared oil. I apply this stain over my relief and ornamental work in the various tints desired. I then rub off the high lights and in general blend in the colors to give it the soft effects. On the plain surfaces I apply the stain in the color desired, then rub off as much as possible; this gives a very pleasing mottled effect that blends in with the under coating of cement paint and takes away the hard appearance.

I have just completed a building in which I used on my relief work blue, golden buff and cream and got a beautiful effect that resembles old bisque china. On some of my work I get these effects by coloring integrally which I rub over with an oil preparation. On residential work my methods are entirely different. I apply the stucco as specified, getting the texture desired preferably, a medium rough cast.

In some cases I color the work integrally, a liquid form of the stucco of the same colors with a binder and hardener and waterproof being added. This material, when properly applied over a fairly rough texture makes a fine finish, and when one gets familiar with its working fine color effects are obtained. This liquid stucco is applied with a brush, same as paint. The stucco surface when finished does not look like paint but retains the softness of the stucco with an unlimited range of color effects. On the ornaments and trim I use color effects with this wash in very much the same method as I specify for my treatment on public buildings except that the material is a stucco composition. To get the shading great care and taste must be used. This liquid stucco coat should be applied before the stucco surface is dry, usually, wherever possible, a day after the stucco is finished. It then

dries and sets along with the stucco and makes a good bond. Spraying with water helps to make the surface bind, using a very fine spray. It gets harder with age and being of practically the same composition as stucco it retains all the soft tints and makes a house very attractive, especially when it has good surroundings. It seems to catch all the shadows and to change with different positions of the sun, reflecting the color of the surrounding foliage. It is this soft color effect that has made the homes of Italy and the south of France the mecca of the students of art. To me, the difference between this treatment and a surface that has been treated with some cement paint is like the difference between a cheap colored lithographic print and a painting. It retains the stucco surface, it keeps out check cracks and avoids the use of artificial paint which are manufactured for this purpose

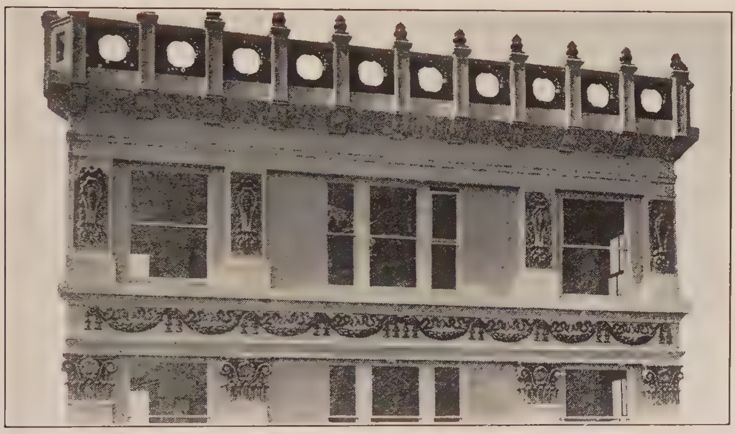

FIG. 2.—SOME UPPER STORY RELIEF WORK IN STUCCO.

but which, while curing, check cracking, gives an artificial appearance and adds considerable to the cost.

The specifications which I give are taken from an article I wrote sometime ago and which covers practically all conditions and treatments except possibly the texture for obtaining the Italian effects. The stucco in this case should not be perfectly straight except in the molding and trim. The molding and trim should be treated as specified, the plain surfaces to give the appearance as if the stucco was applied over cobble stone. No straight edges should be used. The surface should be worked up to a condition with easy modulations. After it is partially set, go over it with the edge of a trowel to slight the roughened surface, being careful not to leave trowel marks. Apply over this the liquid stucco with the desired tints.

I could give my method for mottling the color but I am afraid it might be dangerous; the same applies to the texture. I did a very large stucco job on which I have been working for two years. This was the effect desired.

I had a lot of trouble in getting the texture; the plasterers would either make it too rough or too smooth. I simply had to take a few of my men, train them into the method I wanted; then I chose the ones who seemed to get the idea of the effect desired. These few men I then used on the actual finishing of the work so as it would insure the same texture throughout the entire work.

I have another style of finish which gives a splendid effect and which is very popular. I bring up the work to the straightening coat as specified. For the finish I apply over this surface a very thin coat of cement and sand troweled with a good pressure. I then dash with a whisk broom, being careful not to throw the whole contents of the broom in one place but to spread it and get it with the texture about the size of peas, uniformly over the surface. I then apply over this the liquid stucco coating. A visit to Miami would show the results I have obtained. The photographs which are exhibited do not do the work full justice but they will possibly help to illustrate my methods and the finished results.

SPECIFICATIONS FOR STUCCO ON CONCRETE WORK.

Preparation of Surface.—The entire surface to be examined and all loose form scale removed from the surface (*i. e.*, the scale is caused by cement adhering to forms from previous pours. When the form is not entirely filled in the one day's operations, a film of cement adheres to the form in places and sets when the pour is made. This film invariably forms a scale surface on the face of the concrete when the forms are removed). The entire surface to be gone over with a hand pick or an axe to roughen the surface; if brick, rake out joints. This is for the purpose of forming key for stucco. The surface to be brushed clean and thoroughly soaked, ready for application of stucco.

Proportions: Straightening Coat.—The proportions of this coat shall consist of four parts of portland cement of approved brand, to twelve parts of sand and two parts of hydrated lime, the above materials to be thoroughly mixed dry. Then temper the mortar with water, to which has been added one part of concentrated waterproofing paste to every 25 parts of water.

Finish Coat.—The proportions of this coat shall consist of five parts of portland cement to twelve parts of sand, and 15 per cent of hydrated lime. (If white color is desired, use Medusa white cement and local white sand), the above materials to be well mixed dry. Then temper the mortar with water to which has been added one part of concentrated waterproofing paste to every 18 parts of water.

Application Stucco; Straightening Coat.—Care has to be taken that the surface is thoroughly saturated with water to insure perfect bond, then apply straightening coat. Bring the surface to a true and straight condition, using a traversing rod (no darby float to be used on first coat) then scratch the surface with a wire or nail scratch.

Application of Finish Coat for Smooth Surface.—If stipple, use same process, only stipple before set. If rough-cast dash the finish material

with a broom. Thoroughly saturate the first coat surface with water until it presents a glaze appearance; when this glaze disappears, which will be in a few minutes, apply the finish mortar which should not be too soft and bring the surface to a true condition with darby float. When the mortar will permit, go over the surface with hand float, bringing to a true finish free of cat-faces or voids; the entire surface to be gone over with burlap or hand float and patted to take out float marks. No joints to be allowed in the work where they can be seen. The entire surface to present a uniform appearance in color and texture. Mortar should be applied as quickly as possible and at all times protected from the sun.

Protection.—Special care should be taken to avoid too rapid drying; if in the direct rays of the sun, it shall be protected with burlap or wet canvas, and when sufficiently resistive, should be sprinkled with water for at least six days.

Stucco on Metal Lath.—In stucco on metal lath, specify three-coat work with good fiber in first and second coats. Waterproof in second and third coats.

Forming Molding.—Cores for molding shall be formed of concrete by concrete contractor, allowing about 1 in. for finish. All molding to be run and finished with hand float to give same texture as rest of surface and to help bind the surface. When a condition arises where a heavy coat of mortar is necessary, a key for the mortar shall be formed by driving galvanized nails into the core.

Having thus given specifications for stucco work, it would be well to go over them for the benefit of the craftsman desiring to follow this method of procedure.

The cement, sand, etc., should be well mixed in its dry state and then tempered with water, to which the required amount of waterproofing material has been added. Following this the mixture should be worked to a good plastic condition. In making application, good pressure should be used in order to insure a good bond. In applying the straightening coat, do not use the darby float because the working of this tool is liable to drag the material and interfere with the bond. Rather use the straight edge. Use the rod with an up-and-down slanting motion to cut off the excessive material and leave a rough surface, then scratch with a wire, being careful to scratch before the work is too hard.

After the straightening coat has been applied, the molded and ornamental members should then be worked out. It is common practice with many plasterers to add plaster of paris or some of the patent hard-wall gypsum plasters to the material used for ornamental and run work. This is for the purpose of making the material set quickly, but is a wrong policy and should be avoided. The mechanic who has pride in the execution of his work will not adopt those methods if he knows that bad results will follow by his so doing. Plaster of paris and patent hard-wall plaster are diametrically the opposite to portland cement. The result is easy to foresee. The work blisters,

scales and falls off, the set of the cement is killed, and the material becomes like powder.

The reason that some mechanics use these methods is to gain speed in finishing the work. If, however, the molding and projections are worked together with a little system it will be found that finished results can be obtained with very little more time than through the use of foreign materials. All the moldings and other run work should have running strips set so that the craftsman can build up the various moldings gradually by giving each one as big a coat as will hang, doing the same to the others, and so on. By the time he has gotten to the last piece, he will find the first piece is ready to receive another coat. The running molds should be muffled, allowing about ⅛ in. for finish. After the moldings and run work are brought out to

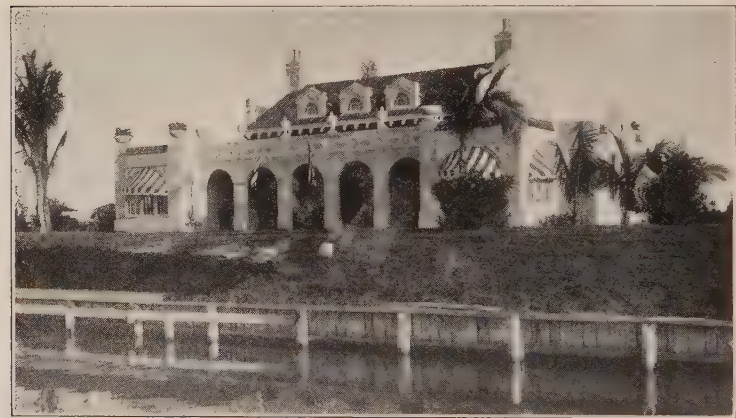

FIG. 3.—STUCCO WORK ON CLUB HOUSE, OCEAN BEACH, FLORIDA.

a complete finish, they should be gone over with floats, so as to insure the surface having the same texture as the rest of the wall.

In applying the finish coat, all splashes and pieces of projecting cement should be scraped off the straightening surface. The whole surface should then be saturated with clean water, and it is at this point that the waterproofing plays such an important part. If the undercoat is waterproofed, it will hold the moisture of the finish coat and will allow the cement its own time to set. The ultimate adhesion, also the uniform working and strength of the finish coat, depends on this. If the moisture, or "blood," of the cement is absorbed by the suction of the undercoat, the cement will become inert and will crack, peel or scale.

In working the finish coat, bring it to a true, straight surface by use of a straight edge and darby float. This material should not be mixed too soft. A good method is to have one set of plasterers lay on the material and then have another set follow with rod and darby, working it carefully

in all directions until it is brought to a full and straight surface. After the moisture has disappeared from the surface, gently scour it with a good cross-grain wood float. Care must be taken if dry spots should develop in floating not to throw water on the wall, but to dampen the float until the desired moisture is showing. Never stop the cement where the joint can be seen. Make all joinings at moldings or projections.

Next, after floating, the object is to be rid of the float marks. This is done by gently patting the surface with a float or by using a pad made of burlap. If the finish is accomplished by this method, good results will always be obtained.

When the work is in the direct rays of the sun, it should be protected with burlap or oil duck cloth hung up in the form of a shade. When the work is sufficiently resistive, it should be kept moist for at least six days.

We have on the market several forms of damp-proof cement paint which make a good preservative of stucco. I advise the use of this material on commercial and public buildings, especially those that are in a large manufacturing center. Such damp-proof coatings protect stucco from the atmospheric chemical action caused by smoke, etc., and at the same time damp-proof, and protect the surface from hair cracks.

For residential work, I advocate the wash method, as it is cheap and can be renewed at very little expense. It artistically weathers with age. It has many more advantages. Among these are the ease with which it can be applied and the wide scope of the colors that can be used. It serves as a good damp-proofing mix and gives the opportunity for many artistic effects. It is the method I use to obtain the Italian and antique effects on stucco. This wash is composed of waterproofing, good mineral colors, cement, and lime with a binder and hardener added. This material has to be used carefully, and the stucco coat surface must be left in a condition that will form a good bond between the materials.

DISCUSSION.

Mr. Collings. MR. W. A. COLLINGS.—I should like to ask Mr. Orr if he has any preference or feels it safer to put his stucco on a galvanized lath or ordinary lath?

Mr. Orr. MR. JOHN C. ORR.—I do not advise the use of galvanized lath that has been galvanized before expansion, in any case. My ideal form of lath is a woven wire lath, galvanized after weaving. A plasterer who can apply it on one can apply it on the other.

Mr. Boyer. MR. E. D. BOYER.—I would like to have the author define what he means by liquid stucco; then I would like also to ask him whether he has had any experience in the use of colored aggregates in stucco rather than pigments, and I should also like him to define what he uses as pigments in securing these colors; whether it is mineral pigment or whether he has any particular preference.

Mr. Orr. MR. ORR.—I make up a liquid stucco with portland cement in the form of a cement wash, to which I add some waterproofing and some hydrated lime and sulphate of zinc. I add the sulphate of zinc to act as a hardener. I have had some experience in colored aggregates in rough-castings. I have taken different kinds of models, roofing tiles and waste from different buildings and dashed them in the form of pebbles and got color effects. I have never used colors as an aggregate. I use mineral colors entirely.

Mr. Boyer. MR. BOYER.—You also referred to cement paint. Describe the difference between liquid stucco and cement paint.

Mr. Orr. MR. ORR.—Cement paint is an oil composition; it is manufactured on the job and I only advise it for public buildings, I do not advise it for residential work, because it gives a painted appearance.

Mr. Boyer. MR. BOYER.—To what extent are you using portland cement for plaster in interior work?

Mr. Orr. MR. ORR.—Only in case of foundries or any place where there is hard wear.

Mr. Boyer. MR. BOYER.—Have you tried portland cement for plastering?

Mr. Orr. MR. ORR.—Yes sir, and got some bad results and some good results. In Great Britain they have plaster cement finished with Keene cement. I know that during my apprenticeship they would not allow you to apply Keene cement over any other material than portland cement. To this day I advocate the same usage in this country.

Mr. Meyer. MR. B. A. MEYER.—Is there any objection to applying liquid stucco with a spray?

Mr. Orr. MR. ORR.—A great deal. You do not get a bond and also you can never get artistic effects with a spray. You get a mechanical effect.

Mr. Collings. MR. W. A. COLLINGS.—I would like to ask Mr. Orr if he gets a shadow effect on the outside by intensifying the dark colors?

Mr. Orr.—Yes sir, you get a shadow effect by that, especially if the modeler does not attempt to smooth up his model. Keep the model rough, getting the darker tints in the background and applying the lighter tints to accentuate the whole figure. Mr. Orr.

Mr. Ralph L. Shainwald.—I would like to ask whether Mr. Orr is not in favor of applying the first coat as soon as possible? Mr. Shainwald.

Mr. Orr.—On metal lath, I apply my first coat on the metal lath and leave that until the largest amount of the interior of the house has been done, so as to provide for any cracking or shrinkage. I leave it just as long as I can; I want whatever cracking or shrinkage there is to come in that coat. On the second coat I use four parts of cement and twelve parts of sand, and apply waterproofing compound with hair in it, and about 20 parts of waterproofing paste. In the finish coat I use 5 parts of cement to 12 parts of sand, and about 10 to 15 per cent of hydrated lime added to each coat. As to cement paint, I put on the wash to insure against the tendency to crack. Mr. Orr.

Mr. Shainwald.—I would like to ask whether, in putting on cement wash, you ever tried mixing fine sand with the cement wash or whether the addition of a small amount of fine material does not even better produce a tendency to wash? Mr. Shainwald.

Mr. Orr.—I put it all through a very thick muslin screen before I apply it. There is an old recipe used in Scotland and England extensively; compose a boil in raw linseed oil, whitening with sulphate of zinc and desired mineral colors to make the tint. That gives a splendid effect and does not show the oil appearance when it dries. It was a common practice in olden times to cover some of the stucco buildings with boiled oils. Mr. Orr.

Mr. Leonard C. Wason.—I would like to ask Mr. Orr if he considers stucco to be equally satisfactory in the North where it is subjected to a very severe change of climate? Mr. Wason.

Mr. Orr.—When the plasterers are educated into the fact that the working of the material requires great care, it will be satisfactory. The climate in Scotland and in Italy alters the same way. Most of the plasterers have neglected their education on stucco. I have 80 plasterers working for me, and I do not get one plasterer in Miami out of 50 who is a stucco worker. Great care should be taken in the North with your joints. It is a common practice on windows for a plasterer to nail a strip on the outside of the window and fill his reveal on the inside, leaving the stucco on until it sets and then he takes it off and applies the other coat. That will never adhere to the previous coat. If he would apply it continuously, avoiding every possible chance of leaving a joint, there is no reason why stucco should not be as successful in the North as any other place. I would not be scared to do it in the North and give a guarantee that it would stay, free from checking. Mr. Orr.

Mr. Cloyd M. Chapman.—I would like to ask Mr. Orr if, over the metal lath, he uses any sheathing board, holding the lath out from the sheathing board and pressing the mortar through against it, or whether he simply presses his lath against the stucco? Mr. Chapman.

Mr. Orr.—I avoid in every possible case the chance of the stucco touching any work. I advise the use of reinforcing iron as a form stretch, Mr. Orr.

Mr. Orr.

to which I attach the metal lath. As a further precaution, I advise the use of a building paper or some other kind of material to keep the stucco from touching the wood. Every time the stucco touches wood, you are going to have a crack.

Mr. Davis.

MR. H. H. DAVIS.—I would like to know whether Mr. Orr's experience has shown whether or not there is a difference in the success of the stucco on concrete or other background over that on a wooden background, that is not necessarily wood lath, but any kind of lath on a wooden frame?

Mr. Orr.

MR. ORR.—You get settlement cracks in it. It can be applied and made just as permanent as any other form of material, but I do not advocate its use to do a job thoroughly over a frame building. It costs so very much less with the use of concrete throughout that I always advocate the use of concrete. You can make a job thorough and get good results from it, but I do not advocate its use at all except in exceptional cases, and in these cases I keep the stucco as far from the wood as I can get it and apply $1\frac{1}{4}$ in. thickness.

ORNAMENTAL PRODUCTS.

By A. G. Higgins.

As artisans in concrete, let us give a few thoughts as to what are ornamental products. My idea is that such products are columns and capitals, urns, railings and balustrades, cornices and cornice ornaments, entablatures, window trim, decorative panels, cartouches, garden furniture, lamp standards, and in fact anything that is used to decorate or ornament a building, cemetery, park or private grounds.

As architects and engineers are the first of the Institute members to have anything to do with products, I will give them my attention first. I find that architects are prejudiced against trying to make anything ornamental of concrete. Why should they be prejudiced? Their particular abomination is the "rock-faced" cement block. Why should they hate the useful block? Because block machine manufacturers consulted an uneducated taste in designing a machine to make a very poor imitation of an excellent natural material. Cheap builders grabbed the block and erected cheap buildings, buildings in which the blocks were so porous that if it rained at Milwaukee cement houses in Chicago wet through and were wet for a week. The ugly block set the standard on which architects based their judgment of the possibilities of concrete products.

Thus the architect has to be enlightened by the products manufacturer. And I will say that architects are some "sot" in their ways. I remember my experience with a very good architect in my city. I called on him every week for several months and the best attention I received was a grunt. Finally, tiring of grunts, I went to his office determined that he would do more than grunt, at least give me a hearing. I stuck until he came to the window, saying, "Old man, what have you got?" I explained the Trusswall column with all the eloquence at my command. He listened, then said, "Why, old man, you cannot make a cement column that is architecturally correct, that has texture, in fact, that looks like anything." My reply was that if he could detail correctly, we could make it and have texture and quality. His pride in knowing all about architectural detail was touched, and he said, "I will make a detail for a model for you, and if you follow the detail and the model has texture, I will specify your column." The model pleased him and he has specified concrete columns ever since and now says he cannot see a column made of anything except concrete.

I believe that most architects have not become interested in the possibilities of concrete products because of the extremely poor quality of work done by many products manufacturers. Also I believe many architects need education and enlightenment as to the possibilities of ornamental products, and would advise them to consult the manufacturer for his suggestions.

* Manager, Trusswall Mfg. Co., Kansas City, Mo.

I find engineers willing and anxious to use ornamental concrete, but their education generally has been sadly neglected in the line of what is architecturally good. As I heard one able architect express it when viewing an extremely ugly bridge structure, "That engineer made that bridge 'hell' for stout, but not much for looks." So I would suggest to the engineer that he employ an architect to clothe his structure, much in the same spirit that most architects now employ an engineer to design the frame for a building.

My advice to cement manufacturers is, do not be too "pinchy" with the products manufacturer. Give him your aid and assistance; he may grow to be a considerable user of cement. Also, improve your product. Especially in regard to the large amount of "dead" matter in your cement.

To the manufacturer of concrete products I would say, improve the quality and workmanship, first remembering that if you are to make clean-looking products that your materials must be clean. The old idea that you could take a bucket of cement and four to nine buckets of dirty sand and rubbish wet up with dirty water and pour the same in a mold and get out a clean-cut piece of work, has been abandoned by the best products manufacturers. Use clean materials, the best you can get, good models that have clean, sharp detail and that are also correctly detailed. And right here, let me suggest to the architect that in designing in concrete he is dealing with a material that is very flexible and he is likely to get the idea that extremely fine lines are desirable. Use judgment, and design somewhat along the lines you would use if you had a lot of money to spend on high-class stone work.

I find many products manufacturers like one I visited in a neighboring city. He took me into the plant and showed me a couple of chunks of concrete he had cast in a sheet-iron mold with dirty sand and dirty water, which he had tried to make into columns. Pointing with pride to the result of his labors he said: "That is what I call a fine column, but them damned architects don't like it." Eventually the architects put him out of business because he did not know enough to consult with them and get their help and suggestions. He still has it in for all architects. Manufacturers, let me suggest this to you. The architect does not have to use your product, but you have to satisfy the architect in order to sell your product to his client. Cater more to the architect if you would increase your business. Architects and engineers generally are ready to cooperate with the products manufacturer if they are assured of intelligent cooperation.

Many products manufacturers complain that there is no market for ornamental products in their territory. Let me ask them seriously, who is to blame? Have they tried to put quality in their products? Has the architect furnished details for the work and have the details been followed? In our plant we have given a lot of time and attention to be sure that the work has been properly executed according to the architect's details, and we believe it pays. Manufacturing of ornamental products requires the closest attention to the smallest detail. Architects are finicky about having a thing properly made, especially after they have spent time and money and thought in detailing. And let me say, they are entitled to just what

they demand in that particular. So it is up to you, manufacturers, to follow details.

There are so many different things to be made of concrete that different methods of making must be used. In making columns, I have found no method equal to the "Trusswall," the patent for which my company owns. The process is about as follows: A core of wood, or better, iron pipe is used upon which to build the column core proper. This core is made of strips of wood nailed to wood circles clamped to the core. Paper is then applied to the core to prevent sticking. A coat of concrete about $\frac{1}{2}$ in. thick applied and allowed to set. It is then wired by revolving the core in the lathe and running the wire back and forth, making a weave. A rough coat is then applied, trued up, set hard, and a finish coat applied. By turning, you get a perfectly round column with no mold marks, and with a proper run board the correct entasis or shape to the column is achieved. This is the only method known to me that makes a column without mold marks, and a column that has good texture and a perfect detail. We also make balusters by the same method.

Ornaments such as enriched cornices, cartouches, classic capitals and modeled work are usually made in gelatin or glue molds. We have made large cartouches where there was only one of a pattern, by modeling in the concrete. This takes considerable skill but is successful if you have that skill. In some cases we use a combination mold, plaster or concrete and glue, using the glue simply for the undercuts and finely modeled parts. We recently made some bracket cross-arms for lamp posts by this method very successfully. Panels of various kinds are made in hard molds with concrete placed semi-dry and properly tamped. This can be done with dry tamp consistency if enough elbow grease is used, but most men have to be watched to have them put enough enthusiasm into their elbow.

Sometimes it is desirable to have a surface that has been tooled or cut after casting in order to give life. Other times, the piece is rough-cast in hard molds and undercut with chisel. In order to do this you must use a mix containing no silica, using marble chips or hard limestone for aggregate. Properly cut, this gives a good representation of stone. In that connection let me protest against trying to imitate stone as a general practice. This reminds me that at one time a visitor to our plant said, "that is a fine-looking stone you are making, what stone are you trying to imitate." I replied that we were not trying to imitate any particular stone, and if any stone happened to look like our concrete work it was not our fault, that we were not interested in imitating anything, but that we believed that concrete had character and quality of its own and we were trying to make the best concrete work we knew how to make.

How is the best way to cure concrete work? I will have to answer that I do not know. After trying as many methods as I could hear of, the problem, to me, still remains unsolved. My belief is, that the first twenty-four hours is the critical time, and that water, vapor or steam have little effect on the hair cracks after that time; that if water can be held in the concrete for

that long, that is, if you allow no evaporation, you will have accomplished all that is possible in that line.

In coloring concrete, several methods can be used. Some soft colors are successfully made by mixing mineral color with the cement. This is apt to have a dull or dry effect. Bright colors may be made with a chemical stain with good results. A successful plan for most work is to use a "cement coat" in colors, such as made by the Glidden Varnish Co. or the Waggoner Paint & Glass Co. and others. This is not a coat made of cement, but a coat made and prepared especially for coating cement work, making it waterproof and drying flat so the texture and character is not lost.

The durability of ornamental concrete is no longer a debatable question, and we are not afraid to put it alongside of stone any time. We have one early piece of work which we like to spring on the doubtful. There are eight fluted columns that are 30 in. in diameter by 25 ft. tall that have been up about seven years. The first story of the building, on which the columns stand, is Carthage stone, the same stone as that used in the new Missouri State Capitol building, and careful examination shows more weathering in the stonework than in the concrete. In fact, I have sent customers out to look at that work and have had them come back and say that they liked the columns very well but that the work under them did not seem to be standing the weather. They had the idea that the supporting work was of concrete. I believe that properly made concrete is more durable than most of the available building stones.

Some of you are interested in the cost of ornamental concrete products as compared with stone or other building material. Possibly our price has been too low, as where we have had a chance to compare, on columns especially, we have been from 40 to 50 per cent of the stone price; equal to 25 per cent above the wood price; from 65 per cent to equal of terra cotta and about the same proportion on other ornamental work. In my opinion, most of us have been afraid to ask a proper price for this kind of work, basing our chances on cheap price rather than on quality. Concrete is not a cheap product, or should not be. In fact, it is a high-class, durable, building material when properly made and should command a proper price.

In conclusion, let me impress on the members that quality is the first requisite for success. That there should be closer cooperation between the different branches of the industry. Pull together and boost every branch of concrete work, lean into the collar and go forward with irresistible force for the best building material on the face of this broad earth.

MANUFACTURE AND SALE OF CONCRETE ROOFING TILE.

By A. P. Tamm.*

Concrete roofing tile is a product that in its short period of existence, has demonstrated its worth. In the construction of factories and mills it finds ready use, as it is economical, inexpensive, permanent, waterproof and above all fireproof. On the home it is used because it has the added advantage of having an attractive appearance. To appreciate these features it must be seen and examined for which reason the discussion in this article will be confined to the process of manufacture with some remarks on its quality and method of sale.

The manufacture of concrete roofing tile is a very simple operation. When made according to the Walter process it requires a special machine that is operated by hand on which an average of from 350 to 400 tile can be made per day. After a few days' time a common laborer can be taught to make this number consistently, some operators exceeding the 500 mark after a little practice.

Tile made in this way are $\frac{3}{8}$ in. thick, $14\frac{1}{2}$ in. long and $9\frac{1}{2}$ in. wide. They interlock on the sides and lap ends, covering a space 8 x 12 in., therefore requiring 150 tile to make one square of 100 sq. ft. Concrete used is made of one part portland cement and three parts clean, sharp sand mixed with enough water to make a semi-wet mixture. This is placed on a cast-iron pallet, set in the machine, whose face is the shape of the back of the tile. The concrete, after being spread over the pallet by hand is struck off several times with a grooved troweling bar until water comes to the top. This bar is shaped so as to form the ridges required in the face of the tile, and produces a highly finished waterproof surface that requires no further treatment, except curing, unless the tile is colored. When a colored surface is desired the coloring matter is distributed over its face with a fine sieve and forced into the body of the tile by troweling.

By means of a foot lever the pallet is next forced through a rectangular frame striking off the edges. It is then removed by hand from the machine and placed on racks where it remains 24 hours. After this time it is taken from the pallet and stacked, being kept in a cool place and sprinkled for ten days. No further treatment is required, but it is advisable to store the tile in the yard for seven or eight weeks before they are again handled. This is not essential, however, as they can be laid with safety after a few weeks if the roofer is careful.

Transportation cost is low because of the light weight of this type of tile. A square weighs but 700 lb., which is very low when compared with other types of roofing material. The concrete tile possess another good feature in that, when stacked, they lie very close together. Close stacking,

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light weight and extraordinary strength permit easy handling which accounts for the low breakage loss they are subject to.

Low cost of laying is another reason why concrete roofing tile are desirable. They are uniform in size and always lay up even permitting them to be placed at a greater speed than clay tile. In speed of laying, wooden shingles cannot compete at all because it takes 900 pieces to cover the same area that is covered with 150 concrete tile. Concrete tile can, therefore, be placed at a greater speed with a consequent reduction in cost of laying.

Cost of manufacture of concrete tile is about \$2.50 per square including material, labor and overhead expenses. They can be laid at a cost not to exceed \$1.50 per square. In this estimate the cost of paper and cleats has been omitted because this is common to all types of roofs. Some builders report that they have laid concrete tile as cheaply as \$1 per square. Accept-

FIG. 1.—PLACING THE TILE ON A ROOF.

ing the estimate of \$1.50, however, and adding to it the \$2.50 cost of manufacture we get a maximum of \$4 per square in place on the roof, a price that, when compared with either some other fireproof roofing material or wooden shingles, is convincing that concrete tile will be the standard roofing material of the future.

At present no particular sales plan has been developed because the demand has exceeded the supply. Architects and builders have sought for a long time a building material that is inexpensive, fireproof, rotproof, waterproof, attractive, permanent and economical. In concrete roofing tile they recognize the material they have been looking for and, as a result, the supply has been unable to keep up with the demand.

Architects, in particular, are glad to have in prospect a material that they can specify, which possesses these advantages without being so high in cost that the owner will not insist upon substituting a cheaper form of construction. Owners, on the other hand, are in a better frame of mind to

listen to the advice of architects and engineers when a more fire-resisting construction is recommended, because they realize that annual fire losses have been unnecessarily high. Engineers find another advantage in the

FIGS. 2, 3.—EXAMPLES OF RESIDENCES ROOFED WITH CONCRETE TILE.

use of concrete tile on account of their lightness. This makes it unnecessary to use an expensive roof frame work in factory and mill buildings.

FIG. 4.—ROW OF HOUSES ALL ROOFED WITH CONCRETE TILE.

Present-day market conditions make concrete roofing tile the best adapted, most economical roofing material. For permanence, economy, fire-resistance and appearance they are unexcelled and for low cost they are in a class by themselves.

DISCUSSION.

The paper was read by Mr. Ferguson of the Hawthorne Cement Products Company.

MR. BELA NAGY.—I would like to know on what is based the cost of $2.50 per square for materials and labor? **Mr. Nagy.**

MR. FERGUSON.—The $2.50 covers the cost of sand, cement, labor, finishing materials and lubricants. You can make the overhead expense whatever you want to; this is the actual cost of manufacturing tile. Depreciation, etc., must be added to that sum. I find that the cost, including light, heat, rent and power runs from $2.19 up. We have a plant in Texas where the cost is very high, being $3.02, but I find the average cost is around $2.50, from 58 plants established in the country. **Mr. Ferguson.**

MR. D. A. ABRAMS.—I would like to ask the speaker if he can give us any information in reference to coloring materials for producing tints in those tiles, what market there is for that kind of material and what success has been obtained? **Mr. Abrams.**

MR. FERGUSON.—The coloring has been looked after by our Service Department. I will say that at the present time they manufacture two colors, the natural cement and the red. Some attempt has been made in the past to use green and we experience a great deal of difficulty in getting a green that will be permanent, one that will be ground in the surface of the tile in the process of manufacture. Some manufacturers buy what is known as an acid proof paint for tile. I do not like that; it will not last over five or six years. Before the present war in Europe, we were able to get a green that has given very satisfactory results, but we can not get it now. We just completed a large building in Indianapolis which is built with stone trimmings and uncolored cement, white in color. I do not believe I have ever seen a more beautiful color combination in my life than this light roof, and while the majority of the people at the present time are buying a colored roof, I believe eventually they will take the normal cement. **Mr. Ferguson.**

MR. RALPH SHAINWALD.—Our company has carried on an investigation of colors for a great many years, and so far as green color for cement products is concerned, the only thing we found to give permanent satisfaction is chromic oxide. The preparations which are sold in the market are efficient just so far as they contain a certain amount of pure chromic oxide. **Mr. Shainwald.**

INSTRUCTION IN REINFORCED-CONCRETE CONSTRUCTION.

By W. K. Hatt.*

Because of the development of reinforced-concrete construction the standard courses of instruction in masonry construction of fifteen years ago have been gradually modified to conform to a new art. Such topics as stone cutting, the oblique voussoir arch, stereotomy, tools for surface finish, various bonds, etc., belonging to the old art, are no longer of much practical interest to the engineering student.

A similar change has taken place in the standard plans of railway structures.

The subject of reinforced concrete has supplied a welcome medium for development of the student's power in the application of applied mechanics, and has also enlarged the field of his use of this study.

The greater complexity of internal stresses demands more intricate analysis, and more refined mathematical tools. The monolithic nature of the construction, whereby beams and slab and column elements are not isolated in reality, demands consideration of continuity and indeterminate action. The classic assumptions of the theory of flexure have had to be critically examined and extended in the case of the combination of two materials, one elastic and one markedly plastic. Indeed, the implications of the phenomena of plastic deformation observed under long-time loads are not yet clear. Likewise, it is a matter of controversy whether or not the well-known analysis of flat plate action for homogeneous flexible materials may be even approximately applied to flat slab floors.

Even a doctrine accepted in the earlier stage of the reinforced-concrete theory, that the action of tensile stresses in the concrete may be omitted from the resisting moment at working loads, has had to be modified for purposes of clear thinking in the light of experimental data.

In a word, with the new freedom in design brought about by this new combination of materials, there has also come a necessity for thoughtful consideration in unaccustomed fields of mechanics, and a lively appreciation of the value of thorough experimental investigations to determine principles of action and an equally lively critical sense with respect to experimental data.

It must be recognized that the concrete constructor has worked in advance of the analyst and experimenter, or logician. The art contains much that is empirical, and conventional. There are cases when construction and logical mechanical analysis are in conflict, and the engineer must decide the relative weight to be given to the logic of mechanics and the instinct of the constructor.

All these matters present problems for the teacher of students in structural engineering.

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The writer holds it as a principle that the mere repetition of conventional designs or mere descriptive matter has little value as the basis for development of the student's mind. These rarely form food for thought. In spite of the intense interest in this novel construction, and the desire of the student to obtain knowledge of approved practice, the subject of reinforced concrete is an especially difficult one to administer from the standpoint of this principle. It is not that the student's interest is not keen enough, nor that the course is not administered with ultimate satisfaction to the class, but that so much of the art is not in shape for instructional purposes. It is difficult to clearly establish the logical series of steps by which a construction is reached. There is bound to be much that is *ex cathedra*, or is justified by a statement of approved practice.

For instance, if one were to merely train detailers, there would be no difficulty in accepting at once a doctrine of non-existence of tensile stresses in reinforced concrete. But if the teacher desires to lay a deep broad basis for subsequent intelligent thought, and approvement of design, he must know the facts and inquire the degree to which this doctrine is useful in the various problems of difficult design or in explaining the phenomena of experiment.

Again the principles of action of web reinforcing are on a very insecure experimental basis, and have little rationality. Such a subject must be presented to the student very carefully and his drawing room work given only after a thorough review of the experimental investigations that establish the present rules with the object of recognizing their limitations.

Again, the subject of reinforced column design requires a very searching analysis of experimental data by the student before the limitations of existing formulæ may be recognized.

In these semi-empirical matters, the instruction should aim at a basis of critical judgment so that in after years the student may be a leader instead of a follower, and will know where to tighten up and where to let loose.

The study of reinforced concrete should follow a general course in testing materials. The student should come into direct laboratory contact with the action of loads, upon reinforced concrete, and perceive and measure resistance under flexural, compression and bond tests as a basis for the mechanics of the subject, and the examination of the more important researches.

The practice in design and proportioning of parts of structures in the drafting room should not, in the author's opinion, extend to such details as lists of dimensions and shapes of all steel bars. Several arrangements should be computed, and the number and disposition of moment and shear reinforcement bars shown. After sufficient practice in computation is had, sizes may be selected from handbooks of standards of conservative companies.

To the usual preparation in applied mechanics must be added the theory of compound stress, of three moments, of the elastic arch, and of earth pressures.

A course of preparation in reinforced concrete as given at Purdue University includes the following elements:

(1) *Materials of Concrete Engineering.*—This is a laboratory course in

the examination and testing of cements, aggregates and the principles of preparations of concrete mixtures, and the technique of testing and measuring including the use of the Berry strain gages.

(2) *A Statistical and Analytical Study of the Extensive Investigations* of the strength of reinforced concrete, such as, for instance, the experiments of Bach, Talbot, Withey and the modern column tests. On the basis of these two elements (1) and (2) the student is able to judge of the meaning of the tests on new forms of construction, such as, for instance, flat slabs and concrete columns with cast iron core, and the standards governing design, and the extent to which the usual formulas for elastic bodies will apply to concrete.

(3) *Principles and Practice in Design of Reinforced-Concrete Structures.*—Having arrived at a thorough knowledge of the physical and mechanical character of materials and of their combined action, the student is then ready to use the principles of applied mechanics in computing sizes of members under load. He must first become acquainted with the usual arrangements of the structural frame in reinforced-concrete buildings and the limitations of sizes of members, sizes of steel bars, influence of the design of centering, and other construction necessities; general construction problems arising from shrinkage, fire resistance, general tying of building together. By descriptive lectures and by reading and exhibitions of actual plans, he becomes acquainted with the thing that he is trying to design. Then follows the application of principles of mechanics, including the imitations of the application to meet the structural peculiarities of the material. Such matters as the position of the moment between positive and negative, to span of beams and computing moment, T-beam action, principles of web reinforcing, must all be explained as logically as possible.

(4) *Study of Laws of Earth Pressure* and the development of some theory such as Rankine's, will lead to the application of reinforced concrete to retaining walls.

(5) *Study of the Elastic Theory* applied to arch design and flat plate design.

(6) *A short Course of Lectures on the Historical* development of reinforced concrete with special application to patents in reinforced concrete.

(7) *Visits of Inspection* to structures under construction.

(8) *In the Drafting Room* the following exercises are completed:

(*a*) Study of typical laboratory test of a reinforced-concrete beam.
(*b*) Test of a reinforced-concrete beam in Testing Materials Laboratory.
(*c*) Study of the reinforced-concrete beam comparing results with theoretical values.
(*d*) Design of reinforced-concrete columns: (1) Longitudinal steel, (2) longitudinal steel tied together, (3) longitudinal steel with hoops.
(*e*) Study of reinforced-concrete formulas noting the effect of change in variables S_s, C_c & P.

(*f*) Study of a typical floor panel using beam and girder system with view of obtaining most economical spacing of beams.
(*g*) Study of architect's plan, selecting the proper arrangement of beams and girders to take care of light, appearance, stair wells, etc.
(*h*) Design of a typical panel: (1) Beam and girder type. (2) T-beam and girder type. (3) Girder and tile. (4) Four-way flat slab. (5) Two-way flat slab.
(*i*) Design of reinforced-concrete footing for building studied.
(*j*) Investigation of a retaining wall: (1) Gravity. (2) Counterfort. (3) Cantilever.
(*k*) Investigation of a bridge abutment holding back an earth fill.
(*l*) Design and investigation of a plain concrete dam.
(*m*) Investigation of a voussoir arch.
(*n*) Investigation of a reinforced-concrete arch by graphical and analytical methods.

The above course extends over a period of twenty-four weeks.

A special elective course for those who expect to enter concrete engineering is given in the senior year. The first semester brings the student in direct contact with original sources, in English, German, French, both of history, theory description and experimental research. Such topics as quality of aggregate, surface finish are reviewed in the library, and reported by the individual student. In short, the report of the Joint Committee is taken as a text and the conclusions examined in the light of records. The more important patents are analyzed and discussed.

In the second semester a particular study is given to the theory and design of concrete bridges, including girder, cantilever truss and arch bridges.

DISCUSSION.

Mr. Allen. Mr. Leslie H. Allen (*by letter*).—It seems to me that one important feature of any course of instruction in reinforced concrete should be a study of the relative cost of the materials and labor entering into reinforced concrete and their influence upon design. We often find a design that is theoretically correct in every particular that would cost many hundreds or thousands of dollars more than another design equally correct which has been made by a man who had some knowledge of the relative costs of concrete, forms and reinforcement.

For instance, we frequently find that a young engineer having figured out the necessary area of steel for the reinforcement of beams will call for a number of different sized bars, such as two $\frac{7}{8}$, one 1, and one $\frac{11}{16}$-in. bar in order to give him the net area, not realizing how difficult it is to get the sixteenth-inch sizes of steel from the mills. Frequently a column is designed with 2 per cent of steel when a richer mix would enable him to stress the concrete higher and make a considerable reduction in the area of steel.

In different localities the relative cost of concrete and steel varies. In New York City where the price of aggregate is very high, it is more economical (within limits) to use a small beam heavily reinforced, but in Buffalo where the price of aggregate is very low, it is economical to use a large beam and cut down the reinforcement as far as possible; the reinforcement costs about the same in either place. Similar considerations make a good deal of difference in the relative cost of flat slab and beam and slab floors.

In designing concrete also the cost of making forms should be taken into account. Where a building of several stories is designed, it is generally economical to make the concrete beams of the roof the same size as those below and lessen the reinforcement rather than change the shape of beams simply because the stresses call for a smaller size.

Such points as these continually come up in contracting work and indicate the need for a careful study by the student of the influence of cost upon design.

AN EXAMPLE OF THE USE OF MOLDED CONCRETE IN LANDSCAPE ARCHITECTURE.

By Linn White.*

By the expression "Molded Concrete" is meant to be conveyed the idea of casting in molds before erection—the opposite of monolithic. Perhaps "Pre-molded" would be more correct, though rather clumsy, as all concrete in a sense is molded.

The work in question is for the improvement and embellishment of a piece of park ground in Chicago about 300 by 1,500 ft., situated in a very prominent position with reference to the center of the city between Michigan Avenue and the Illinois Central R. R., with the Art Institute Building at its southern end and Randolph St. as its northern boundary. It is, in fact, a small slice off one corner of Grant Park which is the lake front of the Loop District.

Grant Park comprises a little over 200 acres, 160 of which lie on the lake-side of the Illinois Central R. R. and 40 acres in the strip between Michigan Ave. and the Illinois Central. It is all made land, as in the beginning Michigan Ave. was practically on the shore line. Tradition has it that the Illinois Central was invited and urged by some of the citizens of the Chicago of those days to build its line along the lake front as it entered the city from the south, so the tracks would serve as a levee or protection against encroachment by the waters of the lake, and that was not much more than a generation ago. Only about 20 years ago Chicago began to wake up to the realization of what a valuable asset its own lake shore is and has been trying ever since to get it away from the Illinois Central.

Some success has been attained in this worthy endeavor. Grant Park has been filled in and is designed to be when completely improved the finest front-door yard possessed by any city in the world. An agreement has been made between the Illinios Central and the South Park Commissioners of Chicago whereby the Park Commission has become possessed of all riparian rights of the railroad, and the dream of the near future is to reclaim from the shallow waters of Lake Michigan an area a half mile wide stretching six miles long from Grant Park southward to Jackson Park. Through the center of this great belt of made land will stretch a lagoon—a protected waterway for all sorts of small craft, spanned here and there by monumental bridges. The full realization of this dream waits upon governmental permits to proceed and upon the settlement of certain vexing questions between the city and the Illinois Central, viz., electrification of all its lines within the city, location of a new passenger station, etc.

* Chief Engineer, South Park Commission, Chicago.

A start on the big scheme, however, has been actually been made with the filling in during the last four or five years of 30 acres of land south of limits of Grant Park upon which is being erected the great Field Museum.

All plans for the improvement of the lake shore start from Grant Park as the center. From Randolph St., the northern boundary of the park, it is proposed to widen Michigan Ave., and extend it in a straight line acress the Chicago River to connect with the lake shore parks of the north, where there is no railroad to complicate the problem of lake shore improvement.

The new bridge across the river will be a monumental structure and will cost, together with the street widening and property damage, approximately $7,000,000. This is an assured project and is now passing through the last legal phases, with every prospect that contracts may be let within comparatively few months.

The prominence of the situation and the intimate connection with other important projects of the improvements intended to be here described were considered justification for the rather unusual and extensive use of ornamental concrete.

It was not desired in this work to make molded concrete that would look like artificial stone or be mistaken for stone. The intention was that it should have a character of its own and appear to be what it is, though it was necessary that the texture and grain of the concrete should be adapted to the architectural lines and classical modeling of the designs.

In general the work consists of a retaining wall about 5 ft. high capped with a balustrade, several flights of steps leading from the lower to the upper level, two pylons flanking the entrance to the bridge across the railroad in the extension of Monroe St., terminal fountains at the ends of Madison and Washington Sts., and a semicircular colonnade with pool and fountain at the northern end of the rectangle facing the Art Institute Building at the southern end.

The foundations for the pylons and colonnade were concrete piles of the Raymond design. Each pile was figured to carry 20 tons. Concrete piles were chosen because the filled earth offered a weak and doubtful support for a spread foundation, and were more economical than wooden piles with the necessity of carrying excavation and concrete down to water level. The concrete piles were topped off at a level just below frost line and monolithic concrete used to construct the foundation up to the ground line. The retaining wall was on a spread foundation, which, in some places where the filled in materials was particularly bad (street sweepings), was carried down to a depth of 11 ft.

Herewith are some photographs of the practically completed structures. It should be understood that the concrete work was just finished at the end of the past season and the surface improvements, such as walks, electric lighting, planting, etc., are not yet complete.

Extracts from the specifications are as follows:

EXTRACTS FROM SPECIFICATIONS FOR ORNAMENTAL CONCRETE WORK IN GRANT PARK.

Concrete.—The concrete used in the various structures shall be of three classes:

No. 1. Foundation concrete.
No. 2. Backing concrete.
No. 3. Concrete for facing or molded work.

No. 1 concrete shall be made of Portland cement one (1) part, torpedo sand three (3) parts, and crushed stone or gravel six (6) parts. It shall

FIG. 1.—GENERAL VIEW OF APPROACHES TO MONROE ST. BRIDGE WITH FLANKING PYLONS.

(Electric lights not completed.)

be used for foundation work and the heavier parts of the bases of the various structures below grade.

No. 2 concrete shall be made of portland cement one (1) part, torpedo sand two and one-half ($2\frac{1}{2}$) parts and crushed stone or gravel five parts. It shall be used for the backing or unexposed body of various structures above the base, such as the wall, curb and bottom of pools and basins, the body of the pylons, columns, pilasters, platforms, steps, etc. None of the No. 2 concrete shall be exposed but all shall be faced with No. 3.

No. 3 concrete shall be made of portland cement two (2) parts, torpedo sand one and one-half ($1\frac{1}{2}$) parts, and limestone screenings four and one-half ($4\frac{1}{2}$) parts. It shall be used for facing all exposed surfaces, for all molded and ornamental portions of the work such as balusters, coping, capitals, bowls of elevated fountains and foliated and modeled parts of the structures. Where used as facing on No. 2 concrete it shall be applied

at least 2 inches thick and shall be placed in the forms at the same time as the other conrcete. Metal separating strips shall be used which shall be lifted as layers of concrete are deposited so the two grades of concrete may be tamped together.

All concrete shall be deposited in layers and well tamped. The layers of No. 1 concrete shall be not over 12 in. thick and the layers of No. 2 concrete not over 8 in. thick. All No. 3 concrete shall be mixed wet so it may be poured and flow readily and shall be carefully settled in the forms.

Mixing.—Mixing of concrete may be by machine or by hand. If by machine the arrangement for measuring proportions shall be positive and accurate and the quantity of water automatically and effectively regulated. The kind of machine to be used must be that approved by the engineer. If by hand the mixing shall be on tight platforms. The sand and cement shall be spread first and turned dry until well mixed, then sprayed with water and the turning continued until a well-mixed mortar is obtained, then the stone added with sufficient additional quantity of water and the whole mass turned until the mixing is complete and regular and the color uniform.

Placing.—The concrete after mixing shall be handled carefully so as not to separate the ingredients and placed in the forms so it may be homogeneous throughout. No concrete shall be retempered or placed after it has attained its initial set. No casting, chuting, or dropping through any considerable distance will be permitted.

Surface Finish.—Special care shall be taken to produce smooth, solid and evenly grained surfaces. To accomplish this end the concrete shall be carefully and evenly mixed with just the right proportion of water and carefully handled by experienced workmen. When the forms are removed the surfaces shall be true, sound and even in grain and color. Inspection on these points will be very rigid, and badly formed or imperfectly finished surfaces will be cause for condemnation of all or any part of any of the structures, although in other respects the concrete may be sound and good. No pointing or patching up of surfaces, lines or corners will be permitted except in some minor cases under the express direction and supervision of the engineer as to the extent to which it may be done and the manner of doing it. The attention of bidders is particularly called to these requirements as a finished and perfect condition of surface is essential and will be insisted on.

After the forms are removed and the condition of the surfaces inspected and approved all exposed surfaces shall be washed with muriatic acid, properly diluted with water, so as to expose the grain and color of the aggregate. Enough acid shall be used of sufficient strength to cut away the surface film of cement but not enough to cause disintegration of moldings, corners, etc. The etching done by the acid must be even and regular over the whole surface so the effect produced is uniform. It shall be done soon after the forms are removed and before the concrete is thoroughly

dry and hard set. The acid shall be applied by hand and the surface scrubbed with fiber brushes. After the surfaces are sufficiently etched they shall be well washed off with water so no acid or discoloration remains.

Granolithic Finish.—The surface of all steps and platforms exposed to traffic shall be finished with a mortar composed of one (1) part Portland cement and three (3) parts fine red granite screenings. The mortar shall be at least one inch thick and shall be applied while the concrete composing the body of steps or platforms is still fresh and has not had time to set. If the steps are molded in blocks the mortar shall be put in the molds at the same time as the rest of the concrete. If the steps are built in place it shall be applied on the top of the freshly tamped concrete and troweled to a finish as in sidewalk work. The surface of the steps and

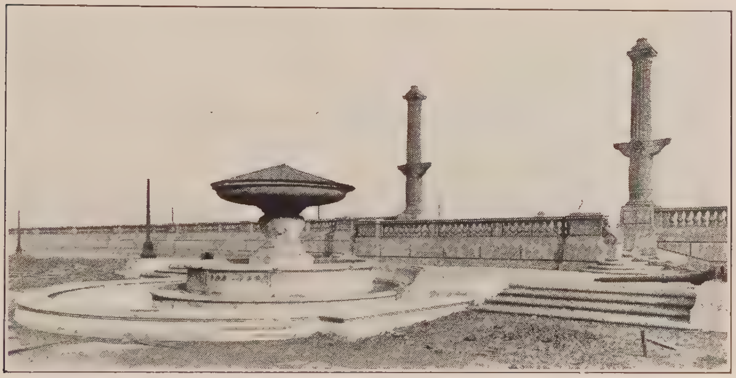

FIG. 2.—VIEW OF THE WALL, BALUSTRADE, STEPS AND TERMINAL FOUNTAIN.

(One terminal fountain is erected at the end of Madison and one at the end of Washington St. The two rostral columns at head of steps will carry ornamental lights. The conical cover on the elevated fountain bowl will be removed.)

platforms shall be washed with acid as provided above for other finished work.

Forms.—The forms may be made of plaster, glue, wood or iron according to adaptability of the materials to the different portions of the work. Whatever material is used the forms must be well and rigidly supported, made in a careful and workmanlike manner and smooth and true in contour. If any preparation is used on the surface to facilitate removal from the concrete it must be non-staining and of such a character as to be easily removed from the surface of the concrete where it may adhere. Forms may be used repeatedly for duplicate portions of the work but must be well cleaned after each use and kept in good condition. Forms

shall remain in place until such time as the concrete shall be sufficiently set to permit their removal without resulting injury to the concrete. Forms shall not in any case be removed until the approval and consent of the engineer has been given for their removal.

Curing Concrete.—All molded concrete must be protected by keeping under shelter, sprinkling, or covering with canvas or other moisture-retaining material. The object is to retard the setting of the concrete and prevent rapid drying out of the surface, which often results in checking or hair cracking, and any reasonable and proper means to prevent this shall be taken as directed by the engineer. Monolithic concrete shall be similarly protected in hot and drying weather, when so ordered by the engineer, by erecting shelters, hanging tarpaulins, sprinkling, etc.

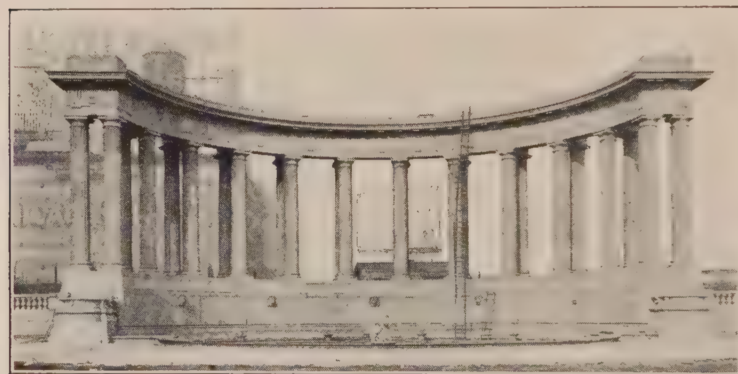

FIG. 3.—FRONT VIEW OF COLONNADE, FOUNTAIN AND POOL.

(Colonnade is about 42 ft. high. Walks and surrounding grading not complete.)

Waterproofing.—All No. 3 concrete used as facing shall be waterproofed by mixing with the concrete approximately ten (10) pounds of some standard waterproofing compound of a make approved by the engineer. The proper method of mixing the compound with the concrete and the proportion to be used shall be as recommended by the manufacturers. The back of the wall and the back of all other structures where one side is backed up with earth and the other side exposed, shall be waterproofed with three coats of bituminous waterproofing, of a character approved by the engineer, with one layer of burlap. The first coat shall be a priming coat put on cold when the concrete is dry and clean. The second and third coats shall be of a consistentcy that required the material to be heated when put on. When the second coat is put on it shall be mopped carefully over the whole surface so every portion is covered, and while still hot and sticky the burlap shall be applied and brushed smoothly

over the whole surface. The third coat of waterproofing shall then be lapped over each other, and over all expansion joints or other joints in the concrete the burlap shall be doubled. The same waterproofing shall be carried through the wall on top of the foundation as a damp proof course and returned up the face of the wall to the surface of the ground. At the top of the wall at the back the edge of the waterproofing layer shall be turned in under the coping which forms the base of the balustrade.

A damp proof course of similar construction shall be carried through the base of pylons, rostral columns, base of colonnade, wall on front line of colonnade, and other walls adjacent to pools where there is danger of moisture being drawn up by capillary attraction and where ordered by the engineer.

Deep and broad grooves shall be formed in the foundation concrete over which the damp proof course is to be laid to form a key to resist sliding of the upper portion of the wall on the damp proof course or dowels may be inserted in the foundation for the same purpose as may be directed by the engineer. .

Sand.—The sand must be clean, coarse lake shore or bank torpedo sand with grains of varying sizes not exceeding 15 per cent passing a 50-mesh screen.

Each bidder must submit sample of the sand he proposes to furnish. The suitability of the sand for the intended purpose, and its resemblance to the sample, will be determined by the engineer, and any rejected sand must at once be removed by the contractor.

Stone.—The crushed stone furnished under these specifications shall be sound, hard, durable, clean limestone free from dust (except in the case of screenings where the dust is included), oil, earthy matter or any foreign substance whatever, either mixed with or coating the stone, that in the opinion of the engineer would render it unfit for the intended purpose. It shall be broken in pieces as nearly cubical in form as practicable. The presence of numerous slivers or spalls, or of numerous soft pieces of stone, will be considered cause for rejection. All the stone supplied for the finished work shall be of uniform color throughout.

For concrete No. 1 the stone shall be of mixed sizes passing a 2-in. screen from which the screenings passing a $\frac{1}{2}$-in. screen have been removed.

For concrete No. 2 the stone shall be that passing a 1-in. screen from which the screenings passing a $\frac{1}{4}$-in. screen have been removed.

For concrete No. 3 the stone shall be screenings passing a $\frac{1}{2}$-in. screen.

Ornamental lighting effects are provided for by rows of concealed lights in the cornice of the pylons and colonnade and in addition flood lighting over the face of the colonnade and fountain beneath is provided by concealed lights with reflectors in the curb of the pool, which is built hollow for that purpose. Lights will be placed beneath glass domes in the elevated bowls of the terminal fountains at the ends of Madison and Washington Sts. so the spray of the

fountain jets rising from the center of the bowls will fall back over the lighted domes. Iron lighting standards (shown incomplete in some of the views) support electric lights for general illumination.

The architectural designs were by E. H. Bennett, Architect, of Chicago; the specifications, engineering features and general direction of the work were by the Engineering Division of the South Park Commissioners, Linn White, Chief Engineer. H. Eilenberger & Co. were the general contractors and the Chicago Architectural Decorating Co., contractors for all molded work.

Committee Reports Presented to the 13th Annual Convention of the American Concrete Institute

REPORT OF COMMITTEE ON REINFORCED-CONCRETE STANDPIPES.

Your Committee on Reinforced-Concrete Standpipes is in the midst of collecting data relating to existing structures, and is not prepared at this time to make a final report.

Your Committee submits the above statement as a progress report and requests that it be continued for another year.

George A. Sampson, *Chairman*,
F. A. Barbour,
Edward Wegmann,
Hiram B. Andrews,
R. B. Tufts.

REPORT OF COMMITTEE ON FIREPROOFING.

As stated in the last annual report, after conferences between the chairman and most of the members of this committee, it was decided that no report should be submitted until the results of at least a considerable portion of the tests about to be undertaken by the Underwriters' Laboratories in Chicago should be available.

None of these tests have been made as yet, but the chairman of the committee is informed that they will begin in the very near future, and it seems possible they will be completed before the next annual meeting.

The same reasons which impelled the committee to postpone a report at the last annual meeting are valid at this time. In view of the great importance of the column in all fire-resisting structures, and of the fact that the tests impending at the Underwriters' Laboratories will constitute the most extensive series of tests of fire-resisting columns ever undertaken, the committee feels that any action without having the results of these tests available would be premature. No doubt, specifications could be drawn which, if adopted and enforced, would bring about an improvement in the methods of fireproofing in all types of buildings. Had the committee undertaken to do this, however, it seems probable that, so far as columns were concerned, the work might easily have proven to be inadequate or out of date almost as soon as it could have been published.

For the above reasons, the committee merely submits this as a report of progress, and requests that it be continued until the results of the tests in Chicago become available.

Respectfully submitted,

John Stephen Sewell, *Chairman.*
Ira H. Woolson,
Edwin Clark,
W. C. Robinson,
C. L. Norton.

REPORT OF COMMITTEE ON REINFORCED-CONCRETE HIGHWAY BRIDGES AND CULVERTS.

In submitting its preliminary report for 1916 the committee confined its discussion to some of the problems relating to design. It has seemed advisable during the current year to supplement this by a brief consideration of some of the more important details relating to selection of materials and supervision of workmanship.

General.

Our highway bridge construction, particularly the shorter span structures, are probably built, as a whole, under less thorough and intelligent inspection than the majority of reinforced-concrete structures, and designs entirely adequate in themselves having their strength and permanence seriously impaired through lax construction methods are not an infrequent occurrence.

One of the most recent failures in concrete highway bridge construction, is a girder span in Polk County, Ore., which collapsed in September, 1916, as the forms were removed. (Figs. 1 and 2.) An examination of this span by one of the committee fully established the fact that incorrect and careless methods of depositing the concrete were chiefly responsible. The frequency of occurrences such as this has seemed to warrant a discussion at this time of some of the points in the selection of materials and supervision of workmanship, lack of attention to which are common causes of failure. The report is comprised under two principal heads: (1) Materials and (2) Workmanship.

I. Materials.

Cement.

A consideration of this topic is hardly within the province of this report. A special committee representing the U. S. Government Departmental Committee, the Board of Directors of the American Society of Civil Engineers, and Committee C-1, on Cement, of the American Society for Testing Materials, in cooperation with Committee C-1, has produced standard specifications and tests for portland cement, effective Jan. 1, 1917, and their recommendations cover the subject thoroughly.

Adequate provision for protection against dampness in storage should be insisted upon. This precaution has received scant attention in many instances known to your committee, particularly in small culvert work.

Fine Aggregate.

Grading.—The density of a mortar and consequently the strength of the resulting concrete will depend to a considerable degree upon the grading of the sand. Undoubtedly the best results will be secured with a sand in which

the various sized particles are carefully limited, but such a sand produced in commercial quantities would, in many instances, be too expensive. For highway bridge construction satisfactory results will be secured with a sand graded from coarse to fine, with the coarser particles predominating. It is desirable to limit the amount of sand passing a 100-mesh sieve to 5 or 6 per cent, and the amount passing a 50-mesh sieve to about 30 per cent. Where the coarse aggregate is screened gravel pebbles it is usually found that the particles of about $\frac{1}{4}$ in. size are in excess. When such a condition exists it has sometimes been found better to limit the maximum sand size to $\frac{1}{8}$ in., especially if the maximum-sized pebbles in the coarse aggregate are less than $1\frac{1}{2}$ in.

FIG. 1.—BRIDGE FAILURE DUE TO POOR WORKMANSHIP.

Improper methods and lack of care in placing the concrete in the columns which support the main girders were responsible for the failure here shown.

When any aggregate is to be used extensively throughout any locality, a study of this point is well worth the expenditure.

Quality.—Clay, loam or vegetable matter in the fine aggregate will coat the particles of sand and weaken the mortar bond. To detect and eliminate material of this nature the silt test should be frequently made. Fine aggregate in which the silt runs above 3 per cent by dry weight is likely to be unfit for superstructure work. A much smaller percentage of vegetable matter (even as low as 0.5 per cent) has been found to produce marked reductions in mortar strength. A recently published series of tests by the Illinois Highway Commission along this line are very instructive.

The value of *routine field tests* on fine aggregates is unquestioned as a

check on the quality. If there is more than 6 per cent settlement after shaking in an excess of water the material represented by the sample should probably be rejected. If there proves to be over 5 per cent, the material should be held pending laboratory tests.

Much of the sand available commercially is full of grains having a coating of limonite or other foreign material. To insure against material of this nature as well as against poorly graded material, a strength test either compression or tension made on 1 : 3 mortar should be employed. No sand should be used which does not develop a strength of at least 100 per cent of the strength of 1 : 3 mortar with standard Ottawa sand, unless the amount of cement is increased accordingly, and in no case should a sand whose strength is less than 75 per cent of the strength of a standard sand mixture be used.

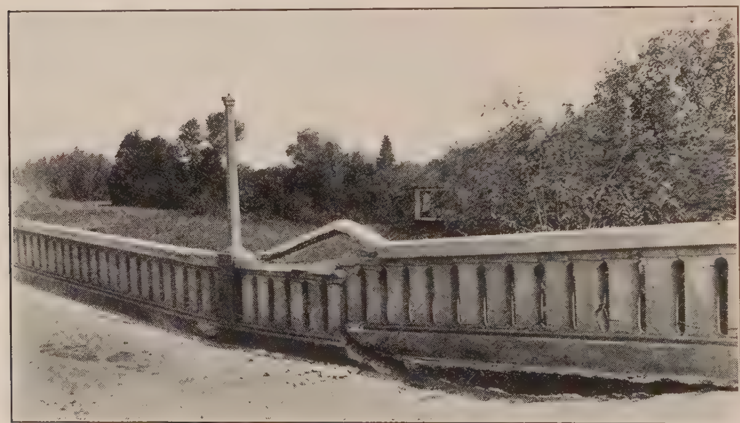

FIG. 2.—VIEW OF STRUCTURE SHOWN IN FIG. 1.

Whenever at all practical the sand used for superstructure work should be washed.

The above tests will regulate the quality sufficiently close for ordinary short span highway bridge construction. For more important work these tests should be supplemented by others such as complete mechanical analysis, voids, specific gravity, mortar density, reaction to litimus, loss on ignition, insoluble silica and microscopic examination.

Gravel or stone screenings between No. 6 and No. 20 mesh may be used if after careful testing no foreign matters are detected that are detrimental to the concrete.

Coarse Aggregate.

Limiting Sizes.—The minimum size is usually specified as ¼ in. while the maximum size will vary with the nature of the construction, size and spacing of the reinforcing bars, etc. In general the maximum size is limited to that

which can be placed around the reinforcing members without danger of forming pockets. General experience limits this size to 75 per cent of the distance from the face of the concrete to the face of the nearest reinforcing member.

The large aggregate is not only the least expensive, but also the least absorbent ingredient, and considerations of quality as well as economy dictate that the amount of mortar be only sufficient to fill the voids as placed. Fig. 3 is the result of tests, by one of the committee, on concrete specimens in which the mortar was maintained at a constant proportion while the amount of stone was varied between 5 per cent and 60 per cent. Porosity of concrete is particularly to be avoided in reinforced work and instances are known to the committee where considerable rusting of the reinforcing bars had taken place

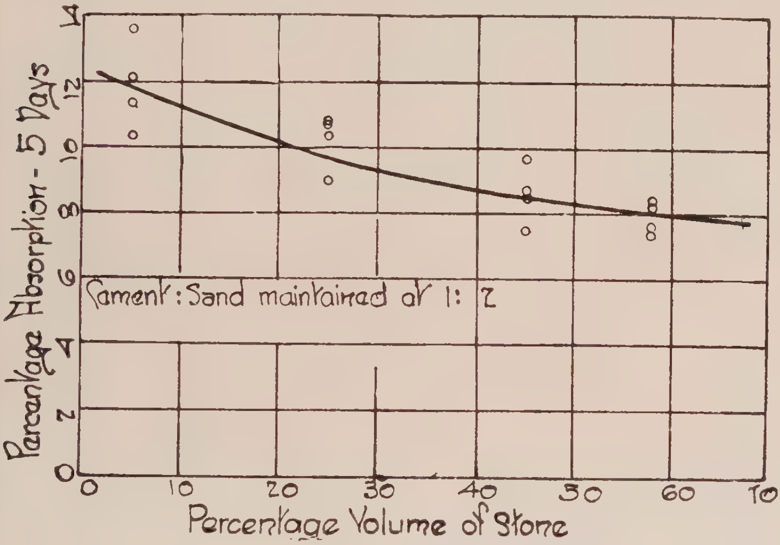

FIG. 3.—VARIATION IN ABSORPTION FOR DIFFERENT PERCENTAGES OF STONE.

due to this cause. For this reason alone careful attention should be paid to the grading of the aggregate and within certain limits the grading that most nearly approaches the curve of maximum density should be obtained. These limits will depend upon the size and nature of the work, the availability of materials, etc. The committee can not make a definite recommendation as to the limits to be used, but does wish to emphasize the importance of attention to this detail.

Broken Stone.—To be suitable for concrete work stone should be reasonably hard and tough and free from soft particles such as shale, slate, mica-schist, disintegrated limestone, or flat or elongated particles. Particular attention should be given to the culling of stone containing particles coated with clay or dust. Many of the quarries furnishing stone for concrete purposes contain ledges of disintegrated material and some of this material

may become mixed with the other and delivered on to the work. A careful inspection of each consignment of stone is necessary to control the quality, and without this any elaborate system of laboratory tests is a waste of time and funds.

Screened Pebbles.—It not infrequently happens that the commercial product obtained under this designation contains a considerable percentage of grains smaller than the nominal minimum size. This may be due to any one of a combination of several causes. Every load of material of this nature delivered should be subjected to a rough screen test and no material accepted which contains over from 5 per cent to 10 per cent (depending upon conditions) under run. Fig. 4 illustrates the fact that certain methods of screening do not by any means remove all the material smaller than the screen size. The four aggregates here shown were screened over $\frac{1}{4}$ in. and $\frac{3}{8}$ in. screens.

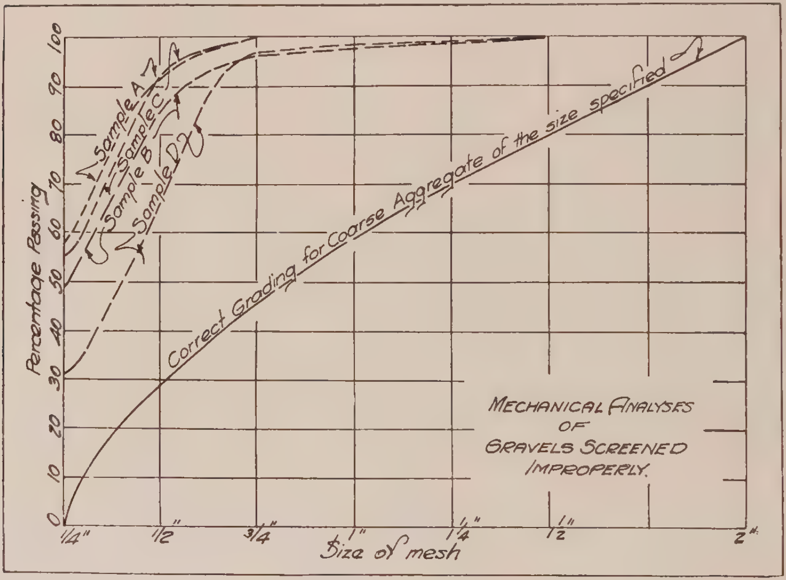

FIG. 4.

Pit-Run Gravel.—Where pit-run gravel is used the ratio of fine to coarse material varies so widely between different deposits and even between different portions of the same deposit that its use as a concrete aggregate leads to a mixture of greatly varying proportions. Consequently as a matter of safety as well as economy such material, on all work of any importance, should be screened to separate the fine and coarse material which should be delivered separately on the job and kept separate until brought together in the mixture in their proper proportions. The best results for pit-run gravel are often obtained by operating a crusher, breaking the over size and screening out the "fines" for use as sand.

REINFORCING METAL.

The committee recommends as a suitable material for reinforcement, steel of a structural grade filling the requirements of the Specifications for Billet Steel Concrete Reinforcing Bars of the American Society for Testing Materials.

Reinforcing metal should be free from rust, paint or grease upon delivery and should be so stored and cared for at the work as to insure their being placed in the structure in a clean condition.

A considerable weakening of the ordinary types of bridge structures, particularly slab and girder construction, may result from the crowding out of place of the reinforcement system during pouring. Special precaution, therefore, should be taken to insure that each bar is securely held to its position by wiring, or blocking to the forms. In general all steel should be securely placed before concrete is poured.

Where bars are spliced, a length of lap sufficient to develop the full strength of bar should be used. Transfer of stress by means of bond should not occur in regions of maximum tensile stress in the concrete. If clips of an approved design or other method of rigidly connecting the bars of a reinforcing member be used, the length of lap may be decreased an amount depending upon the individual nature of the connections.

Reinforcement built into rigid frames is recommended for slab and girder construction whenever practicable.

FORMWORK.

All forms should be built of material sufficient in strength to hold the concrete without bulging between supports. In selecting this material, therefore, a knowledge of the pressure of the green concrete against the forms becomes essential. Experimental data along this line are somewhat meager.

It would seem that for large massive construction when concrete is poured rather slowly the forms may in all safety be designed for an equivalent fluid pressure of 85 lb. per cu. ft. When the sections are light, as in viaduct construction, concrete rapidly poured may exert a fluid pressure as great as 150 lb. per cu. ft. against the forms, some recent tests indicating a value even greater than this. Concrete falling against formwork may result in high lateral pressure due to impact.

Forms should be filleted at all sharp corners and in case of all projections such as girders, copings, etc., the forms should be given a bevel or draw sufficient to insure their easy removal.

Forms should be built tight to avoid the loss of cement through seepage, should be dressed smoothly wherever the concrete which is placed next them is to form an exposed face.

The committee recommends that in general forms be painted with boiled linseed oil or other equally good preparation to prevent the concrete adhering to the same. Crude oil or any material such as tar paper that will stick to, or discolor the concrete should not be used.

II. WORKMANSHIP.

Care of and Delivery of Materials.

In dry weather aggregates may be stored on the gound without becoming mixed with the same. In wet weather serious results are certain to originate in this practice. It is especially hard to remove the lower portions of a pile of *fine* aggregate without including some of the material of the road bed which has become more or less mixed with it. The extra expense of a platform for such material is amply justified.

PROPORTIONS AND PROPORTIONING.

The proportions specified will vary with the type of construction, with the availability of the various materials of construction and with the type of design. In general the following proportions are representative of common practice:

Class I. (For superstructure work and for reinforced cantilever type substructure.)
- 1 part cement,
- 2 parts sand,
- 4 parts broken stone or screened pebbles.

Class II. (For walls of gravity abutments and piers.)
- 1 part cement,
- $2\frac{1}{2}$ parts sand,
- 5 parts broken stone or screened gravel.

Class III. (For footings.)
- 1 part cement,
- 3 parts sand,
- 6 parts broken stone or screened pebbles.

Class IV. (For thin section slabs and girders.)
- 1 part cement,
- 2 parts sand
- $3\frac{1}{2}$ parts broken stone or screened pebbles.

These proportions are not by any means a fixed standard, but may be varied within limits to suit local conditions. When once established, however, every precaution should be exercised to maintain uniformity in the mix. There are many conditions that make this difficult and uncertain.

Where the coarse and fine aggregates are piled too near each other there is danger that the edges of the two may overlap and thus material from the bottom of either pile will be more or less mixed with the other. This is not an uncommon occurrence in small span bridge work.

The method of proportioning by shovels is open to serious question. Uniformity of proportion can only be maintained by the use of bottomless boxes or wheelbarrows whose contents have been carefully pre-determined.

The relationship between the amount of mixing and the uniformity of the resulting product is strikingly brought out in Fig. 5. Lack of uniformity means uncertainty as to structural strength for which reason the reliability

of concrete is dependent on no other factor more than thorough and adequate mixing. Inasmuch as it is difficult to determine by visual inspection whether concrete is thoroughly mixed it is recommended that specifications be drawn to prescribe a definite period of time. The recommendations of the Joint Committee cover this phase of the subject quite fully.

The most effectual method of mixing is by machinery, and experience has amply demonstrated that a batch mixer of the best type should be used. For work of any size it is the part of wisdom to carefully proscribe the type of mixer to be used. The following requirements for mixing machinery are recommended.

I. It is desirable to have the mixer equipped with an attachment for

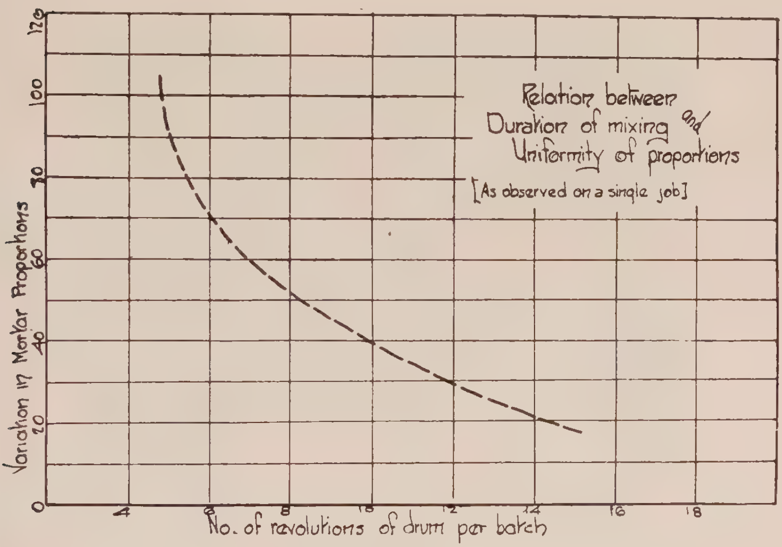

FIG. 5.

locking the discharging device so as to prevent discharging a batch before the same has been mixed the required length of time.

II. The mixer should be equipped with means for preventing the addition of aggregate after the mixing has commenced.

III. The mixer should be equipped with water storage and a measuring device which can be locked.

IV. The mixer should be equipped with a batch meter or other device for accurately recording the number of revolutions of each batch. An *autographic* device is to be recommended.

V. The speed of the mixer should be carefully regulated to give at the periphery of the drum the speed best suited to the batch. Speeds between 175 and 225 ft. per minute seem to give the best results.

Insufficient mixing will result from operating the mixer above its rated

capacity. The maximum size of batch for each size and type of mixer in use on the work should be accurately predetermined and the batch limited to this amount.

If hand mixing is necessitated the same should be done on water-tight platforms and special precaution taken after the water has been added, to turn all the ingredients until the mass is homogeneous in appearance and color.

PLACING CONCRETE.

Even though the concrete is accurately proportioned and thoroughly mixed the method used in placing may be such as to segregate the materials

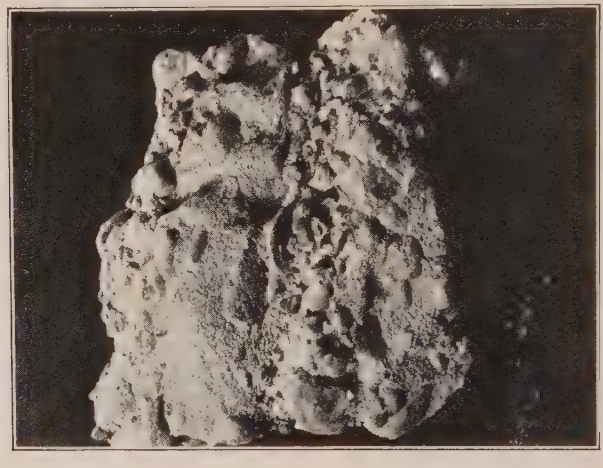

FIG. 6.—SPECIMEN OF POOR CONCRETE.

Showing the stratification of materials and the excessive formation of laitance in concrete deposited without due precaution to avoid separation of the ingredients.

and materially lessen the strength. Fig. 6 is of the concrete taken from a portion of the columns under the bridge shown in Fig. 1 and 2, and shows the marked segregation that may occur from a careless manipulation of the materials.

When concrete is deposited through a spout or open trough the angle of the same with the horizontal should be such as to allow the concrete to flow without separation of the ingredients. About one vertical to two horizontal is good practice. There are several features that should be emphasized in this connection.

The spout should be kept clean and free from lumps of hardened concrete by thoroughly flushing with water before and after each run. The spout should extend as near as possible to the point of deposit. Where the

discharge must be intermittent a hopper should be provided at the bottom for distribution. Spouting through vertical pipes should be checked by baffle plates when the flow is discontinuous. Where concrete is to be placed under water the greatest care is necessary to prevent segregation and the consistency should be carefully regulated. When closed chute or tremie methods are used the greatest care must be exercised to avoid the formation of excessive laitance and the stratification of the materials as shown in Fig. 6. The following suggestions relative to this point may be emphasized.

1. The concrete should be mixed with more water than is ordinarily permissible so that it will flow readily through the tremie and into place with a nearly horizontal surface.

2. The coarse aggregate should be smaller than ordinarily used to facilitate the flow.

3. The mouth of the tremie should be buried in the concrete so that it is at all times entirely sealed. The concrete thus discharging without coming in contact with the water.

4. The tremie should be suspended so that it can readily be lowered when necessary to choke off or retard the flow.

The flow should be continuous in order to avoid the formation of laitance in the interior. In case the flow for any reason is interrupted, *special care* should be exercised to entirely remove all laitance before proceeding with the work.

Other methods of depositing concrete under water, such as the bottom dump bucket, are sometimes used. In any case it is essential that still water be maintained at the point of deposit and that any condition favorable to a separation of the ingredients be strictly guarded against.

CONSISTENCY.

The use of too much mixing water causes a segregation of materials, decreases the early strength and renders the concrete porous. The finely divided cement and clay or silt from the aggregate is washed to the surface by the excess water, and cleavage planes of laitance are formed. Moreover the excess water forms a film surrounding each aggregate particle and since the consistency is not such as to admit of further consolidation, this water is not forced to the surface, but remains to impair the bond between aggregate and cement. When concrete deposited too wet is spaded next to the forms the cement is drawn to the surface in many cases leaving air and water voids beneath the surface. Fig. 7 is a photomicrograph of a concrete section showing an elongated air void of this nature.

Freezing and thawing, expansion and contraction, etc., will break loose the surface scale leaving a surface such as shown in Fig. 8. A consistency too wet is thus frequently the cause of surface trouble in concrete bridge work.

Concrete deposited too dry on the other hand will be hard to handle and inclined to form pockets around reinforcing bars. The amount of mixing water should be carefully regulated at all times.

CURING OF CONCRETE SURFACES.

The top surfaces of all walls, abutments, girders, slabs, handrailings and copings should be carefully tamped and given a smooth even surface by the use of a suitable finishing tool. Such surfaces should be protected from the sun and sprinkled in dry weather and the whole surface kept wet for a period of at least one week. Floors should preferably be covered with damp earth and the same kept wet for at least ten days or until a thorough curing of the concrete is insured.

FIG. 7.

Photomicrograph (magnified 40 diameters) showing an elongated air void surrounding a sand grain caused by the presence of a film of "free water" which originally surrounded the sand grain. The consistency and consolidation was not such as to force this water to the surface before the concrete had attained its initial set.

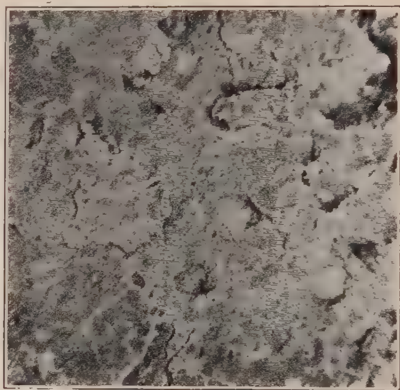

FIG. 8.

A concrete surface which has scaled badly, in this case the result of a consistency so wet that the finer particles including most of the cement were washed against the forms "robbing" the portion immediately underneath.

SURFACE WORKING.

To eliminate the formation of stone pockets not only at the surface, but also around the reinforcing system, the concrete should be thoroughly worked and spaded as it is deposited.

REMOVAL OF FORMS.

The forms under superstructures should be left in place for at least three weeks in warm summer weather and longer in cold weather. In removing them extreme care should be exercised not to damage corners of the green concrete, and any rough places or holes which appear should be immediately patched with a mortar of the same composition as that used in the concrete.

FILLING AGAINST WALLS AND ARCHES.

Earth filling against abutment, wing and spandrel walls should be placed in horizontal layers avoiding wedge-shaped sections against the walls. Arch fillings should be deposited in such a manner as to load the arch symmetrically. The filling back of abutments and between spandrel walls should be thoroughly drained by means of drainage lines and scuppers through wing walls, arch rings and piers.

FIG. 9.

Showing the typical formation of laitance on the surface of a concrete. To avoid seams of this character in the finished work it is necessary to require that each morning, before any concrete is run, the old surface be removed to a depth sufficient to expose the coarse aggregate.

DRAINAGE OF FLOORS.

The floors of all slab and girder bridges should be provided with suitable drain scuppers at frequent intervals.

COFFER-DAMS.

Coffer-dams should be constructed as near water-tight as practicable. No pumping should be permitted inside of foundation forms while concrete is being placed, and should it become necessary to prevent flooding a seal of concrete may be deposited in the bottom of the coffer-dam and allowed to set. This concrete must be deposited through a closed chute in the most approved manner. Arch foundations on piling should be enclosed in permanent continuous sheet piling, with the tops sawed off one foot below low water.

BONDING SUCCESSIVE LAYERS OF CONCRETE.

As nearly as practicable construction joints should be made perpendicular to the lines of least tension and in such location as to be least noticeable. Before any new concrete is placed the old surface should be thoroughly cleaned of all dust, dirt or laitance and treated with a cement paste wash. If there are no reinforcing rods projecting small stóne should be one-half imbedded in the old concrete to form a bond. Fig. 9 shows the character of surface presented when concrete is deposited too wet. To avoid seams of this character it would seem necessary to require that each morning before any concrete is run the old surface be removed to a depth sufficient to expose the coarse aggregate, and then prepared as above. Special precaution should be taken to avoid improper cleaning of concrete in foundation pits after they have been flooded.

EXPANSION JOINTS.

Where expansion joints are shown on plans special precaution should be taken to insure a complete separation of substructure and superstructure.

SURFACE FINISH.

All exposed surfaces should have a smooth and neat appearance, and the general appearance of the work can be greatly improved by such special finishes as scrubbed surfaces, bush hammered or tooled surfaces, etc., the special finish being applied to such portions of the surface as may be determined by study of the particular design in question.

CONCRETE IN FREEZING WEATHER.

Concrete should not be mixed or deposited at a freezing temperature, unless special precautions are taken to prevent the use of materials covered with ice crystals or containing frost and to prevent the concrete from freezing before it has set and sufficiently hardened. In general all concrete should be maintained at a temperature of not less than 40° F. for at least 48 hours. As the coarse aggregate forms the greater portion of the concrete it is particularly important that such material be warmed to well above the freezing point. The lowering of the freezing point by the use of salt, calcium chloride or other anti-freezing materials is not to be recommended.

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REPORT OF COMMITTEE ON CONCRETE AGGREGATES.

Your Committee on Concrete Aggregates is still carrying on its investigations in cooperation with Committee C-9 of the American Society for Testing Materials. The field includes in its most important studies: Laws of Proportioning Aggregates; Methods of Testing Coarse Aggregates; Methods of Testing Fine Aggregates; Impurities in Sand and Remedies Therefor; Methods of Manufacture and Test of Laboratory Specimens of Concrete; Method of Manufacture and Test of Field Specimens; and Size and Shape of Test Pieces.

The researches have not yet reached a stage for definite reports, with the exception of the tests of weight given below, but it is expected that before the Convention next year conclusions of interest will be brought out.

Determination of Weight per Cubic Foot of Sand.

One of the investigations made under the joint auspices of the two committees and specifically under the direction and supervision of the Subcommittee on Weight, Voids, and Specific Gravity, of which Mr. Cloyd M. Chapman is Chairman, has been of a series of tests on the weight per cubic foot of sand under different conditions and by different methods.

The following laboratories participated in this test:

The New England Bureau of Tests.
Westinghouse Church Kerr & Company.
The Pittsburgh Testing Laboratory.
The N. Y. Public Service Commission Laboratory.
The Columbia University Laboratory.
The Lehigh University Laboratory.
The Office of Public Roads of the U. S. Department of Agriculture.

The object of the test was to determine the method and apparatus by which the most suitable results may be obtained in the determination of the weight per cubic foot of sand by different operators under different conditions.

The variables which need to be standardized are as follows:

(1) The size and shape of the measure.
(2) The method of filling the measure.
(3) The amount of moisture in the sand.

The sands used were of two grades, a coarse and a fine sand. The coarse sand showed not over 15 per cent passing a 50-mesh sieve. The fine sand showed not over 20 per cent retained on a 30-mesh sieve. Both of these sands were tested while dry and while moist. The dry sand was dried at room temperature until it flowed freely through the fingers, and did not contain more than 1 per cent of moisture. The damp sand had enough water added to it to bring the moisture to about 5 per cent in the case of the fine sand and about 3 per cent in the case of the coarse sand.

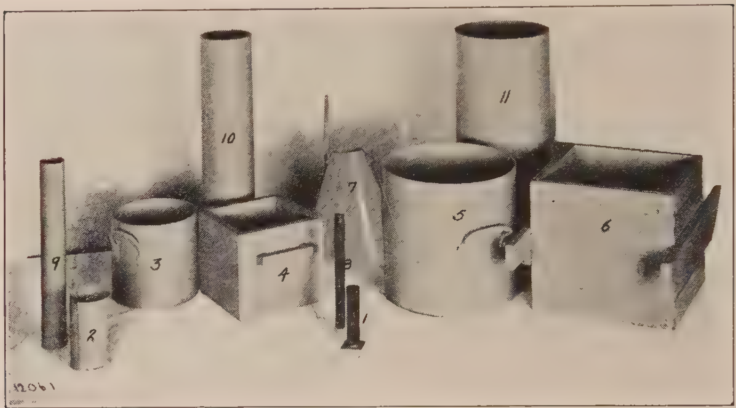

FIG. 1.—MEASURES AND TUBES USED TO FILL THEM.

The sand was tested each day for the percentage of moisture before the tests were started for that day. Care was taken that these percentages were approximately maintained throughout the tests, water being added from day to day as the tests proceeded in order to take the place of that lost through evaporation.

DESCRIPTION OF APPARATUS.

In the following description the numbers in parenthesis refer to the location, in photographs herewith (Figs. 1 and 2).

Four sizes of measures were used, having capacities, respectively of 100 c.c., 1000 c.c., ¼ cu. ft. and 1 cu. ft. The 100 c.c. measure was cylindrical and was

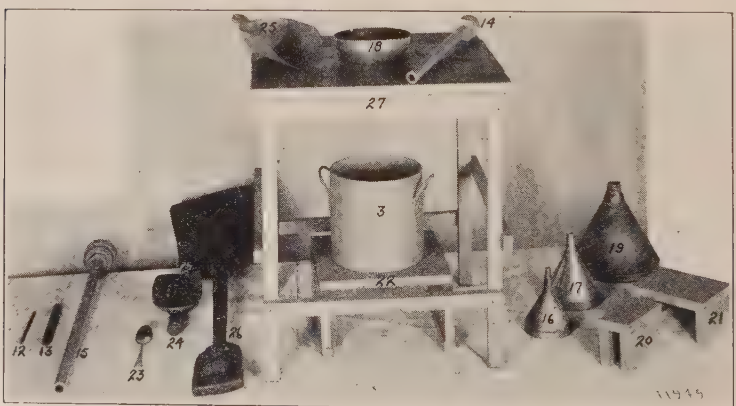

FIG. 2.—ADDITIONAL APPARATUS USED TO FILL MEASURES.

about $1\frac{1}{4}$ in. diameter by $5\frac{1}{2}$ in. high, (1). The 1000 c.c. measure (2) was cylindrical with a diameter of $3\frac{1}{2}$ in. and height of $6\frac{1}{2}$ in. The $\frac{1}{4}$ cu. ft. measure was made both cylindrical and cubical. In the case of the $\frac{1}{4}$ cu. ft. cylindrical measure (3) the diameter was 8 in. and the height $8\frac{1}{2}$ in. The $\frac{1}{4}$ cu. ft. cubical measure (4) measured $7\frac{9}{16}$ in. each way. The 1 cu. ft. measure was also made both cylindrical and cubical. In the case of the former (5) the diameter was 13 in. and the height 13 in., while the latter (6) was 12 in. each way.

The 100 c.c. and 1000 c.c. cylindrical measures were made of galvanized sheet iron with galvanized iron bottom, and the $\frac{1}{4}$ cu. ft. and 1 cu. ft. cylindrical measures were made of iron pipe cut to proper lengths with metal bottoms. The $\frac{1}{4}$ cu. ft. and 1 cu. ft. cubical measures were made up either of substantial sheet metal or well waterproofed wood.

Later a fifth size measure was used. This was a truncated cone (7) made of No. 16 gage galvanized iron with a base soldered on and the top open. The diameter of the base was 10 in., the height 10 in., and the diameter of the opening in the top 3 in. Its capacity was 0.2066 cu. ft.

The seven measures were all accurately weighed, and the five metal measures had their cubic capacity determined by weighing the quantity of water required to fill them level full. The two wooden measures had their cubic capacity determined by carefully measuring each of their twelve edges.

For use with these measures there were four metal tubes, open at both ends, having capacities respectively of 110 c.c., 1100 c.c., 0.3 cu. ft. and 1.1 cu. ft., these capacities being approximately 10 per cent in excess of the capacity of the measure with which they are used in Method G, described below.

These tubes were made up with the following approximate dimensions:

110 c.c. tube (8) 1 in. diameter by 11 in. long.
1100 c.c. tube (9) $2\frac{1}{4}$ in. diameter by $17\frac{1}{2}$ in. long.
$\frac{3}{10}$ cu. ft. tube (10) 6 in. diameter by $16\frac{3}{4}$ in. long.
$1\frac{1}{10}$ cu. ft. tube (11) 10 in. diameter by 24 in. long.

There were also four tampers for the four capacities of measures of the following description:

For the 100 c.c. measure a $\frac{1}{4}$ in. round iron rod (12) 8 in. long.
For the 1000 c.c. measure a $\frac{7}{8}$ in. round iron rod (13) 12 in. long.
For the $\frac{1}{4}$ cu. ft. measure a tamper (14) having a circular flat head 2 in. in diameter with a handle from 18 to 24 in. long, the whole tamper weighing 3 lb.
For the 1 cu. ft. measure a tamper (15) having a circular flat head 3 in. in diameter with a handle from 2 to 3 ft. long, the whole tamper weighing 5 lb.

Four funnels were provided, having openings respectively $\frac{3}{8}$ in. (16), $\frac{5}{8}$ in. (17), $\frac{7}{8}$ in. (18) and $1\frac{1}{4}$ in. (19) in diameter, for filling the measures. Each funnel was provided with a rigid support for holding it in a fixed position above the measure it was used to fill.

There were also three wooden stands (20, 21, and 22) of various heights used in conjunction with the support for the funnels. One of these was

placed under the measure according to its height and insured the distance from the bottom of the funnel to the top of the measure being as called for in method of filling F.

For filling the measure the following apparatus was also used:

For the 100 c.c. measure a large spoon (23) or tablespoon.

For the 1000 c.c. measure and $\frac{1}{4}$ cu. ft. measure metal scoops (24 and 25) as shown in the photograph accompanying.

For the 1 cu. ft. measure a shovel (26) as shown in the photograph, and lastly, one or more metal straight edges for striking off tops of measures when full.

In Fig. 2 are shown the table (27) for supporting the funnel when filling the measure, and the $\frac{1}{4}$ cu. ft. round measure (3) in place on the stand (22) provided for it, with the scoop (25), tamper (14) and funnel (18) used in filling this measure.

The seven different methods of filling the measures described are designated by letters, namely, A, B, C, D, E, F, and G. The instructions given each laboratory were as follows:

Method A—Filled and Struck.

Set the measure on a firm level surface. Fill with the sand, using spoon, scoop, or shovel, depending on size of measure, taking several quantities of sand from different parts of the sample. Fill the measure to overflowing and without jarring or moving it, strike off the surplus sand from the top with a straight edge. Place measure on scales and weigh.

Method B—Filled and Jarred.

Fill measure to overflowing by putting in several portions with spoon, scoop, or shovel. Jar measures in the following manner: In the case of the 100 c.c. and 1000 c.c. cylinder, tip the measure about 10° off the vertical and jar by lifting and dropping about one inch, 25 times, allowing the measure to strike on the outer edge at the bottom, and rotating the measure during the process, so the sand moves from one side to the other in the measure. In the case of the $\frac{1}{4}$ cu. ft. and 1 cu. ft. measures, jar by rocking from side to side 25 times, then fill to overflowing, strike off and weigh.

Method C—Jarred while Filling.

Fill measure one sixth full, jar 15 times as described in method "B," add another sixth and jar again in a similar manner. Repeat until full. After last jarring fill to overflowing, strike off and weigh.

Method D—Tamped in Layers.

Fill measure one-sixth full and tamp 20 times with proper tamper, using short quick strokes. Add another one-sixth and repeat till full. Then fill to overflowing, strike off and weigh.

METHOD E—FILLED AND TAMPED.

Fill measure with spoon or shovel and tamp 25 times with proper tamper, using firm strong strokes, fill to overflowing, strike off and weigh.

METHOD F—FILLED THROUGH FUNNEL.

Support funnel rigidly over the middle of the top of the measure with the bottom of the outlet at the following heights:

	IN.
For 100 c.c. measure	2
For 1000 c.c. measure	3
For $\frac{1}{4}$ cu. ft. measure	5
For 1 cu. ft.	$7\frac{1}{2}$

This will allow the sand to pile up in a cone above the top of the measure without reaching the bottom of the funnel. Fill the measure by placing sand in the funnel, always keeping a steady flow from the mouth of the funnel. Always have sand in the funnel, so that the flow will be uninterrupted, till measure is full. In making the damp sand tests, a small wire constantly moved about in the mouth of the funnel will keep the sand flowing. Keep the wire in the neck of the funnel and continually moving while the measure is being filled. Fill measure to overflowing, strike off and weigh.

METHOD G—INNER TUBE.

Stand the tube, holding 10 per cent more than the measure, upright in the center of the measure. Fill the tube with sand. Without moving or jarring the measure, slowly lift the tube at a speed of about 1 in. per sec. out of the measure, allowing the sand in the tube to flow from the bottom of the tube and fill the outer measure. Strike off the surplus sand from the top and weigh.

CONE METHOD.

Fill the cone and, holding a piece of waste over the opening in the top to prevent sand being shaken out, rock violently to and fro about thirty times. Fill with sand where it has settled down and repeat. Continue this until there is no settlement. Strike off and weigh.

This last method was decided on after the others were completed and was only done in the laboratory of Westinghouse Church Kerr & Co. In order to make up for the lack of different laboratories participating, five different operators were used on each grade of sand, both wet and dry and five determinations were made with each. The results of this last method have been added to the tables and surves compiled from the previous methods, and will be found under the heading of the word "Cone."

In all cases, except the cone method, each operator made five tests with each of the seven methods of filling, each of the four sizes of measures, using each of the two grades of sand, both wet and dry, making a total of 560 determinations of the weight per cu. ft. Where it was not found possible to complete the test in this manner, either one of the grades of sand, one

of the four sizes of measure, or one of the seven methods of filling were omitted, but in all cases five tests were made with each method, each measure, and each sand, where these tests were made at all.

Care was taken that the apparatus used conformed to the description given to the various laboratories that participated; also that the various operators carefully and accurately followed the methods prescribed. Wherever possible, the tests were made by an operator experienced in this kind of work in order to get more accurate results.

In reducing the data gathered in the foregoing tests to a more useful form and to produce this data graphically, three lines of reduction were pursued. These three lines will be referred to hereafter for the sake of brevity as (1) Average weights, (2) Average maximum variation and (3) Average mean variation.

The first of these is self-explanatory. The second is the difference between highest and lowest weights obtained by all laboratories with each size of measure, each method of filling and each grade of sand. The third is the difference between each of the five individual results obtained by each laboratory, and the grand average of all laboratories then averaging these five differences to get the mean variation.

Taking the first of these lines of reduction we have Table and Curves I. This is obtained by averaging the weights obtained by all the laboratories with all sizes of measures.

Table and Curves II show the average weights obtained by all methods of filling.

In the second line of reduction Table and Curves III show the average maximum variation obtained by all laboratories using all sizes of measures.

Table and Curves IV show the average maximum variation for all laboratories by all methods of filling.

In the third line of reduction Table and Curves V show the average mean variation for all laboratories for all sizes of measures.

Table and Curves VI show the average mean variation for all laboratories for all methods of filling.

Table and Curves VII are recapitulations of Tables and Curves I, III, and V, by taking from each of these tables and curves section "b," omitting however lines and curves showing "difference" between wet and dry sands.

In an exactly similar manner Table and Curves VIII are recapitulations of Tables and Curves II, IV, and VI.

AVERAGE WEIGHT BY DIFFERENT METHODS (TABLE AND CURVES 1).

This table is derived by averaging the six weights obtained with six sizes of measures for each kind of sand both wet and dry, except the cone method where only one size of measure and one method of filling was used. The third line in the table is the difference between the 1st and 2d lines and shows the difference in the average weight obtained with dry and wet sand of the same kind, that is, these figures represent the difference due to dampness. The 6th line is the difference between the 4th and 5th lines. The

9th line is the difference between the 7th and 8th lines. The last line is the average of the 1st, 2d, 4th and 5th lines.

These curves show the relation existing between the method of filling the measure and the resulting weight per cu. ft., for both dry and wet sands, and also the difference between the weight with dry and wet sands.

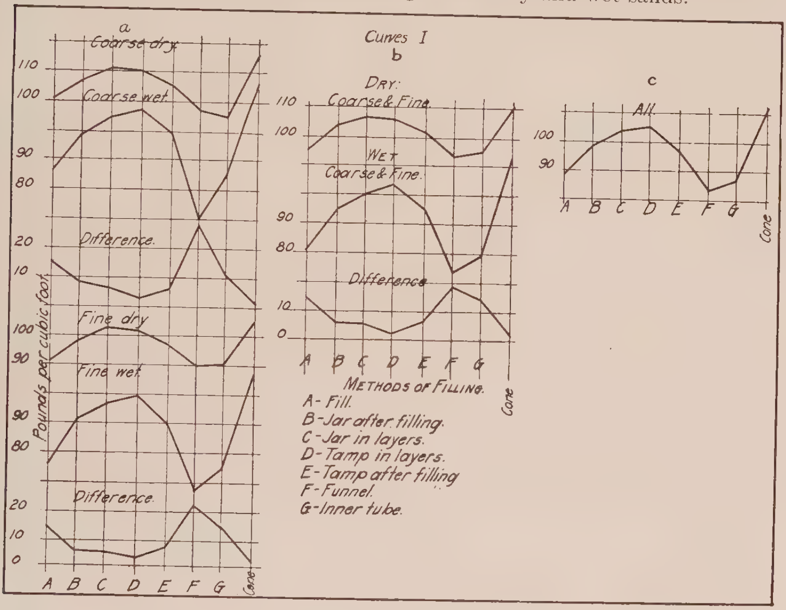

TABLE I.—AVERAGE WEIGHTS OF ALL LABORATORIES WITH ALL SIZES OF MEASURES.

Sand.	Method.	A	B	C	D	E	F	G	Cone.
a 1 Coarse	Dry	100.5	107.4	110.7	110.3	105.4	97.6	95.5	115.28
a 2 Coarse	Wet	85.8	99.1	104.0	107.3	99.1	69.6	84.4	117.18
a 3 Difference	Coarse	14.7	8.3	6.7	3.0	6.3	28.0	11.1	−2.02
a 4 Fine	Dry	90.9	98.1	102.4	102.3	97.5	89.7	90.5	105.75
a 5 Fine	Wet	76.3	90.8	95.9	99.0	90.2	67.5	75.2	108.22
a 6 Difference	Fine	14.6	7.3	6.5	3.3	7.3	22.2	15.3	−2.47
b 7 Dry	Coarse and fine	95.7	102.7	106.6	106.3	101.5	93.7	94.7	110.52
b 8 Wet	Coarse and fine	81.1	95.0	99.9	103.1	94.6	74.2	79.8	112.76
b 9 Difference		14.6	7.7	6.7	3.2	6.9	19.5	14.9	−2.24
c 10 Average of all		88.4	98.8	103.2	104.3	98.0	83.9	87.2	111.64

Attention is called to the markedly greater effect of method of filling when wet sand is used than when dry sand is used, as indicated by the greater difference between the high and low points on curves for wet sand than for curves for dry sand.

The lesser effect of method of filling is an argument for the use of dry

sand in making weight determinations. A peculiarity in the relative effect on weight of method F and G should also be noted, namely, that with damp sand method F gives much lower weights than method G, while with dry sand the two methods give about equal results. A similar relation exists between methods C and D, namely, that while methods C gives slightly higher results than method D, when dry sand is used, yet with wet sand method D gives noticeably higher results than method C.

In the curves showing difference between weights obtained with dry sand and with wet sand by the same method, it should be noted that these curves are almost complements of the curves above them showing weight per cu. ft. In other words, the method giving the highest weight per cu. ft., (cone method) shows the least difference between the dry and wet results, and the method giving the lowest weight (method F) shows the greatest difference between the dry and wet results; and that the other methods are arranged in exactly the same order in the "difference" curves that they are in the "weight" curves.

It is also worth remarking that the cone method gives by far the highest weight both wet and dry with both grades of sand, also that it is the only method in which the weight of wet sand is higher than the weight of dry sand. This is very noticeable on the accompanying curves.

AVERAGE WEIGHT WITH DIFFERENT MEASURES (TABLE AND CURVES II).

This table is derived by averaging the seven average weights obtained with the seven methods for each kind of sand, both wet and dry. The 3d line in the table is the algebraic difference between the 1st and 2d lines, and shows the difference in the average weights obtained with dry and wet sand of the same kind, that is, these figures represent the difference due to dampness. The 6th line is the algebraic difference between the 4th and 5th lines. The 9th line is the algebraic difference between the 7th and 8th lines. The last line is the average of the 1st, 2d, 4th and 5th lines.

These curves show the effect of the size of the measure used upon the weight per cubic foot for both dry and wet sands, and also the difference between the weight dry and the weight wet with the same size of measure.

Attention is directed to the increase in weight per cubic foot as the size of the measure is increased except with the cone method, which consistently gives much higher weights per cubic foot than any other measure used. With the exception of the cone method, however, this increase is true, and to a greater extent with wet sand. Dry sand gives nearly uniform weights with all sizes of measures, and except in the case of the cone method, the largest measure averages less than 2 lb. more than the smallest measure. The wet sand shows considerable difference due to the size of measure, the largest measure averaging about 12 lb. per cu. ft. higher than the smallest measure. Attention is also called to the curve showing difference between the weight of dry sand and wet sand with the same size of measure, by which it is clearly indicated that this difference is much more marked in the case of small measures than with large measures, and also that there is practically the same difference with round measures as with cubical measures. The startling

feature is again supplied by the cone method, wherein the wet sand weighs more per cubic foot than the dry sand, this being the only measure which gives such results.

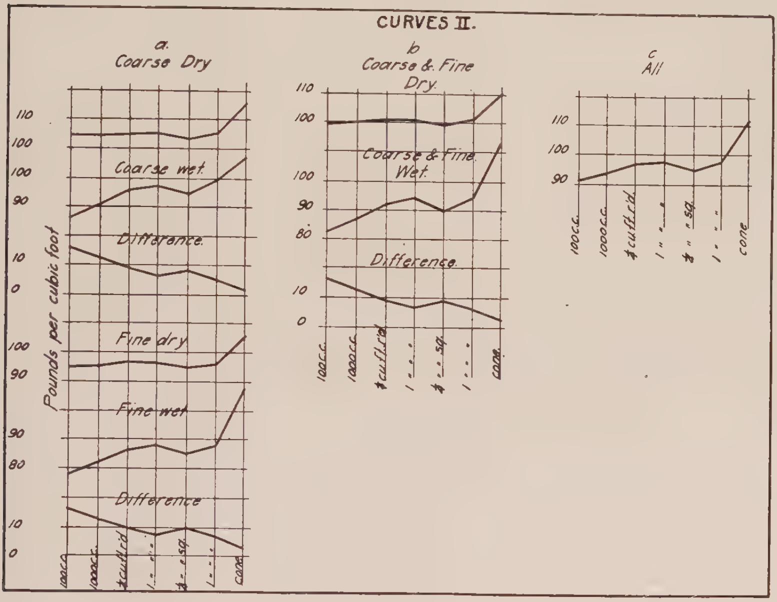

TABLE II.—AVERAGE WEIGHT OF ALL LABORATORIES WITH ALL METHODS.

		Sand.	Measure.	100 c.c.	1000 c.c.	¼ cu. ft. rd.	1 cu. ft. rd.	¼ cu. ft. sq.	1 cu. ft. sq.	Cone.
a	1	Coarse........	Dry..........	104.1	104.1	104.5	104.9	103.7	105.0	115.28
	2	Coarse........	Wet..........	87.0	91.0	95.8	98.0	95.1	99.4	117.18
	3	Difference.....	....	17.1	13.1	8.7	6.9	8.6	5.6	−2.02
	4	Fine..........	Dry..........	95.0	95.7	96.9	96.4	94.9	96.6	105.75
	5	Fine..........	Wet..........	78.0	82.5	86.9	88.4	85.5	88.6	108.22
	6	Difference.....	....	17.0	13.2	10.0	8.0	9.4	8.0	−2.47
b	7	Dry..........	Coarse and fine	99.6	99.9	100.7	100.7	99.3	101.3	110.52
	8	Wet..........	Coarse and fine	82.5	86.8	91.4	93.2	90.3	94.0	112.76
	9	Difference.....	....	17.1	13.1	9.3	7.5	9.0	7.3	−2.24
c	10	Average of all..	....	91.1	93.3	96.0	96.9	94.8	97.4	111.64

The curves show that if sand is weighed dry the capacity of the measure has little effect upon the result, except in the case of the cone method, but if weighed wet, small measures give lower results than large measures and if the sand is weighed wet, the large measure gives results more nearly approaching the dry weight than small measures. It must be borne in mind that the

cone method is the only one in which a container or measure is used, wherein the sides are not parallel, but slope in towards the top, naturally tending to pack the material, more than in measures with vertical sides.

DIFFERENCES BETWEEN HIGHEST AND LOWEST WEIGHTS FOR DIFFERENT METHODS (TABLE AND CURVES III).

This table is derived by taking the difference between the highest and lowest weights obtained by all laboratories with each size of measure, each method of filling and each grade of sand. Section ''a'' of this table is obtained by averaging the six results obtained with six sizes of measures for each kind of sand, both wet and dry. Section "b" is derived from section "a" by averaging the 1st and 3d, and 2d and 4th lines respectively. Section "c" is the average of lines 1, 2, 3 and 4, of section "a."

The curves show the effects of method of filling on the maximum range of results obtained. Attention is directed to the much greater maximum range shown for the wet sand than for the dry sand. This is one argument in favor of using dry sand in making weight determinations. It should also be noted that the cone-method gives less range of results than any other method, and that method D is second in this respect.

DIFFERENCE BETWEEN HIGHEST AND LOWEST WEIGHTS FOR DIFFERENT MEASURES (TABLE AND CURVES IV).

This table is derived by taking the differences between the highest and lowest weights obtained by all laboratories with each size of measure, each method of filling and each grade of sand. Section "a" of this table is obtained by averaging the seven results obtained by the seven different methods of filling. Section "b" is the average of the 1st and 3d and 2d and 4th lines respectively of Section "a." Section "c" is the average of lines 1, 2, 3, and 4 of section "a."

The curves show the effect of the size and shape of the measure upon the maximum range of results obtained.

Attention is directed to the general lower range of the larger measures than of the smaller measures. The difference due to size and shape of measures is not great, but it is fairly consistent, and is somehat greater for the cubical measures than for the cylindrical measures. The cone method shows a lower range of results for both grades sand than any other measure.

VARIATIONS FROM THE MEAN BY DIFFERENT METHODS (TABLE AND CURVE V).

This table is derived from the difference between each of the five individual results obtained by each laboratory and the grand average of all laboratories, then averaging these five differences to get the mean variation. Section "a" is obtained by averaging the six averages obtained with the six sizes of measures for both coarse and fine sands wet and dry. Section "b" is the average of the 1st and 3d and 2d and 4th lines respectively of Section "a." Section "c" is the average of lines 1, 2, 3 and 4 of Section "a."

The curves show the relation between method of filling and variation from the mean weight for the various sands.

Attention is called to the low variation from the mean of method D and the cone method for both dry and wet sand, also that with dry sand the variation due to method of filling is very much less marked than with wet sand. It is also shown that with dry sand either method D or E gives equally low

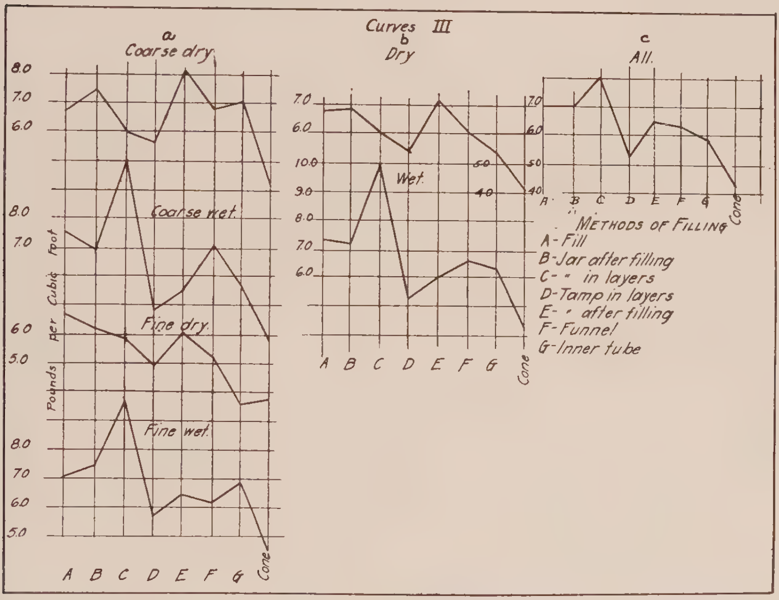

TABLE III.—AVERAGE OF DIFFERENCE BETWEEN HIGHEST AND LOWEST WEIGHTS OBTAINED BY ALL LABORATORIES WITH ALL SIZES OF MEASURES.

		Sand.	Method.	A	B	C	D	E	F	G	Cone.
a	1	Coarse......	Dry....	6.75	7.40	6.03	5.67	8.10	6.77	7.05	4.27
a	2	Coarse......	Wet....	7.52	6.88	9.98	4.78	5.53	6.98	5.65	3.84
a	3	Fine.........	Dry....	6.72	6.23	5.93	4.92	6.08	5.30	3.60	3.74
a	4	Fine.........	Wet....	7.10	7.40	9.80	5.65	6.40	6.18	6.92	4.61
b	5	Dry.........	....	6.74	6.82	5.98	5.30	7.09	6.04	5.33	4.00
b	6	Wet.........	....	7.31	7.14	9.89	5.22	5.97	6.58	6.29	4.23
b	7	Difference...	....	0.57	0.32	3.91	0.08	1.12	0.54	0.96	0.23
c	8	Average of all	....	7.03	6.98	7.94	5.25	6.53	6.31	5.81	4.12

variation from the mean, although method D is one in which high weights per cubic foot are given, while method F is one which gives low weights per cubic foot, as shown in curves of Tables I and II, also that the cone method shows the least variation from the mean of any method.

Again with wet sand the least variation is shown with the cone method, with method D standing second.

VARIATIONS FROM THE MEAN WITH DIFFERENT MEASURES (TABLE AND CURVE VI).

This table is derived by taking the difference between each of the five individual results obtained by each laboratory and the grand average of all laboratories, then averaging these five differences to get the mean variation.

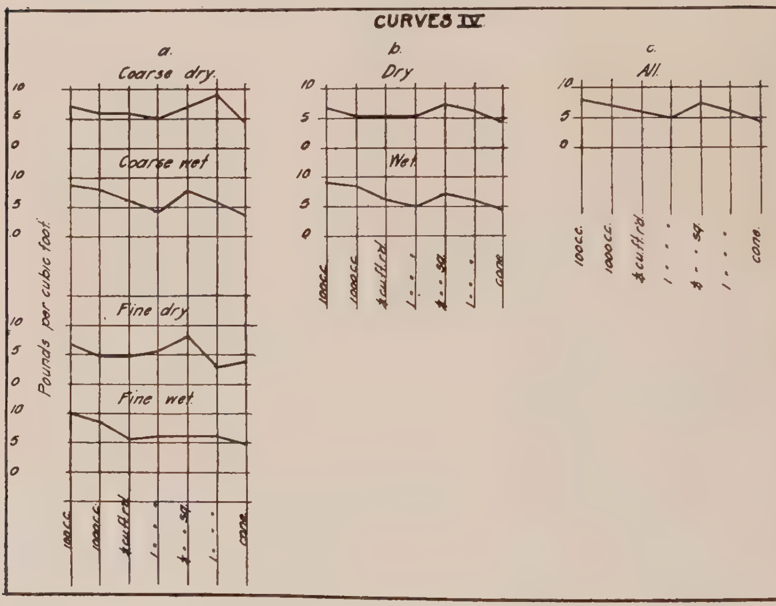

TABLE IV.—AVERAGE OF DIFFERENCE BETWEEN HIGHEST AND LOWEST WEIGHTS OBTAINED BY ALL LABORATORIES BY ALL METHODS.

Sand.			Measure.	100 c.c.	1000 c.c.	¼ cu. ft. rd.	1 cu. ft. rd.	¼ cu. ft. sq.	1 cu. ft. sq.	Cone.
a	1	Coarse	Dry	7.1	6.2	6.3	4.9	6.8	9.4	4.27
	2	Coarse	Wet	8.6	7.8	6.5	4.2	7.6	5.8	3.84
	3	Fine	Dry	6.7	5.0	4.8	5.6	8.0	2.9	3.74
	4	Fine	Wet	10.0	8.5	5.7	6.0	6.0	6.1	4.61
b	5	Dry		6.9	5.6	5.6	5.3	7.4	6.2	4.00
	6	Wet		9.3	8.2	6.1	5.1	6.8	6.0	4.23
	7	Difference		2.6	2.6	0.5	0.2	0.6	0.2	0.23
c	8	Average of all		8.10	6.88	5.83	5.18	7.10	6.05	4.12

Section "a" is the average of the seven averages obtained by the seven methods for both coarse and fine sand, wet and dry. Section "b" is the average of the 1st and 3d, 2d and 4th lines, respectively, of Section "a." Section "c" is the average of the 1st, 2d, 3d and 4th lines of Section "a."

The curves show the relation between size and shape of measure and average variation from the mean.

It should be noted that with cylindrical measures the larger ones give less variation from the mean than the smaller ones, and that the larger cubical measures are subject to greater variation in results than the larger cylindrical

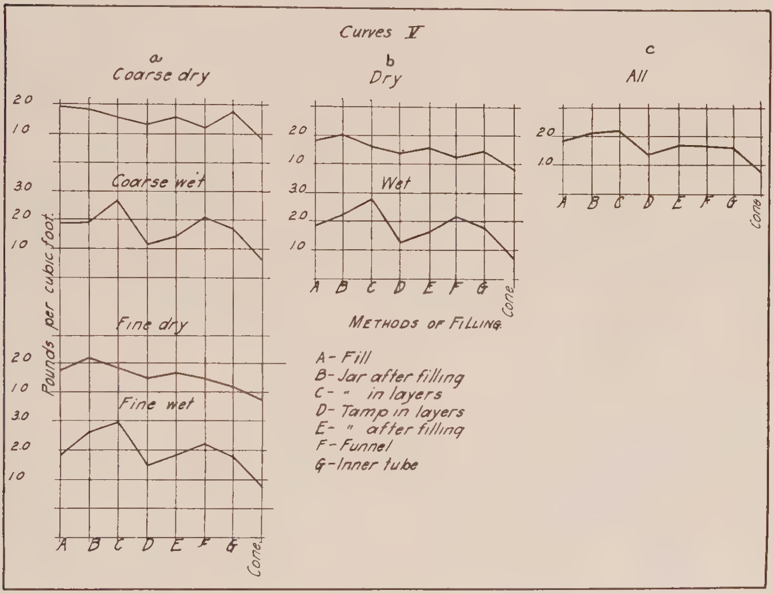

TABLE V.—AVERAGE FOR ALL LABORATORIES FOR ALL SIZES OF MEASURES OF THE VARIATION FROM THE MEAN WEIGHT OF ALL LABORATORIES.

	Sand.	Method.	A	B	C	D	E	F	G	Cone.
a	1 Coarse......	Dry....	1.96	1.84	1.53	1.33	1.60	1.23	1.82	0.92
a	2 Coarse......	Wet....	1.93	1.90	2.67	1.22	1.39	2.08	1.76	0.70
a	3 Fine........	Dry....	1.75	2.16	1.80	1.48	1.67	1.52	1.17	0.86
a	4 Fine........	Wet....	1.84	2.59	2.94	1.54	1.85	2.23	1.86	0.80
b	5 Dry........	....	1.86	2.00	1.66	1.41	1.63	1.29	1.50	0.89
b	6—Wet........	....	1.88	2.24	2.81	1.38	1.66	2.16	1.81	0.75
b	7 Difference...	....	0.02	0.24	1.15	0.03	0.03	0.37	0.31	0.14
c	8 Average of all	....	1.87	2.10	2.19	1.41	1.71	1.69	1.64	0.82

measures. Also that the cone method gives very much lower results in average variation from the mean than any of the preceding methods.

These curves would indicate that cylindrical measures are to be preferred, and that about $\frac{1}{4}$ cu. ft. is sufficient size to give best results, *i. e.*, most concordant results.

EFFECT OF METHOD OF FILLING ON AVERAGE WEIGHTS AND VARIATIONS (TABLE AND CURVES VII).

This table is derived from Tables I, III and V, by taking section "b" from each of these tables and combining them in this table, but omitting curves showing "difference" between wet and dry sands.

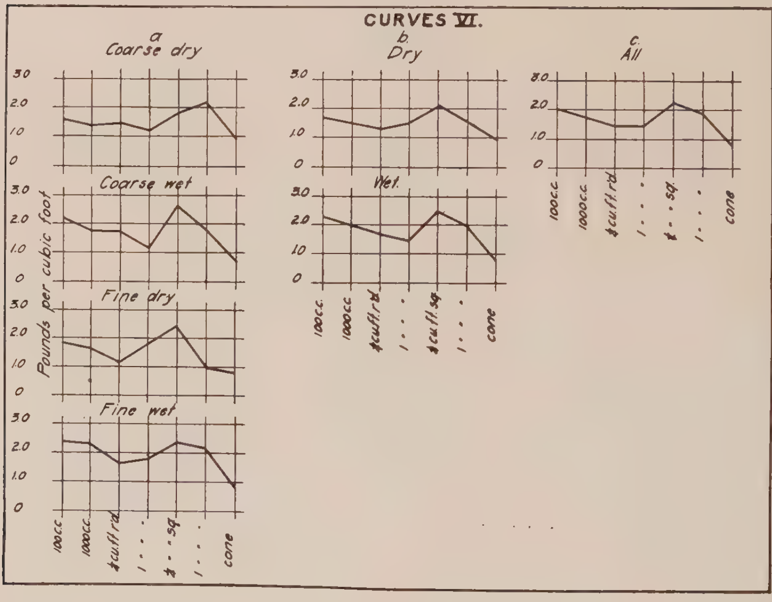

TABLE VI.—AVERAGE FOR ALL LABORATORIES FOR ALL METHODS OF THE VARIATION FROM THE MEAN WEIGHT OF ALL LABORATORIES.

	Sand.	Measure.	100 c.c.	1000 c.c.	¼ cu. ft. rd.	1 cu. ft. rd.	¼ cu. ft. sq.	1 cu. ft. sq.	Cone.
a	1 Coarse	Dry	1.63	1.38	1.45	1.23	1.81	2.17	0.92
a	2 Coarse	Wet	2.19	1.75	1.74	1.12	2.58	1.80	0.70
a	3 Fine	Dry	1.93	1.65	1.17	1.78	2.62	1.05	0.86
a	4 Fine	Wet	2.39	2.29	1.65	1.81	2.38	2.22	0.80
b	5 Dry		1.73	1.52	1.31	1.51	2.12	1.61	0.89
b	6 Wet		2.29	2.02	1.70	1.47	2.48	2.01	0.75
b	7 Difference		0.56	0.50	0.39	0.04	0.36	0.40	0.14
c	8 Average of all		2.01	1.74	1.47	1.49	2.25	1.92	0.82

The curves show the relation that exists between method of filling the measure and average weight per cu. ft., average maximum variation or the difference between highest and lowest weights in each laboratory, and average variation from the mean, for wet and dry sands.

A study of these curves shows that for maximum weights per cu. ft. the cone method should be used, while for minimum weights per cu. ft. method F should be used, the other methods ranging in between these two extremes.

As regards average maximum variation, the cone method gives the least difference between highest and lowest results for both wet and dry

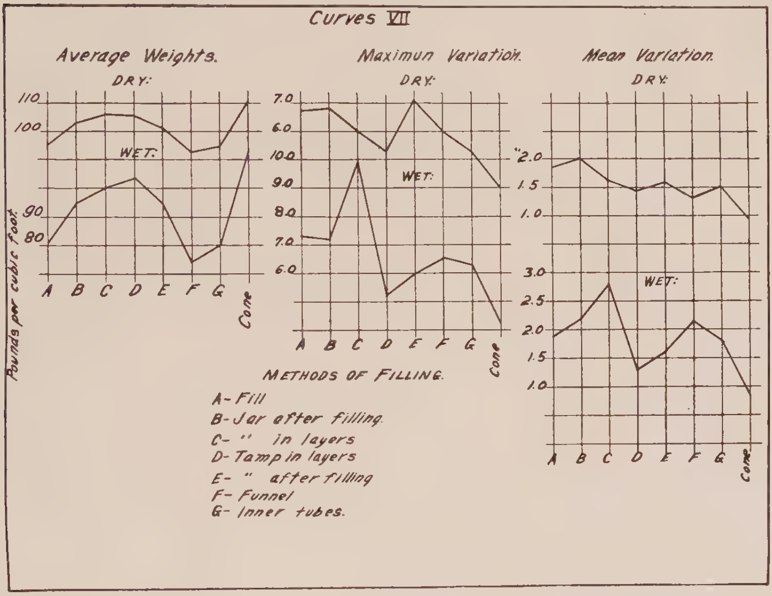

TABLE VII.—EFFECT OF METHOD ON AVERAGE WEIGHT, AVERAGE VARIATION FROM MEAN WEIGHT, AND DIFFERENCE BETWEEN HIGHEST AND LOWEST WEIGHTS OF ALL LABORATORIES, FOR ALL RESULTS.

Method.	A	B	C	D	E	F	G	Cone.
Average weight	88.4	98.8	103.2	104.3	98.0	83.9	87.2	111.64
Average maximum variation	7.03	6.98	7.94	5.26	6.53	6.31	5.81	4.12
Average variation	1.87	2.10	2.19	1.41	1.71	1.69	1.64	0.82

sands, while method C is most conducive to large variations for wet sands and method E shows largest variations for dry sands.

In the matter of average variation from the mean it is the cone method again which gives most concordant results and methods C and E give least concordant results for wet and dry sands respectively.

EFFECT OF MEASURE ON AVERAGE WEIGHTS AND VARIATIONS (TABLE AND CURVES VIII).

This table is derived from Tables II, IV, and VI, by taking section "b" from each of these tables and combining it in this table, but omitting curves showing "difference" between wet and dry sands.

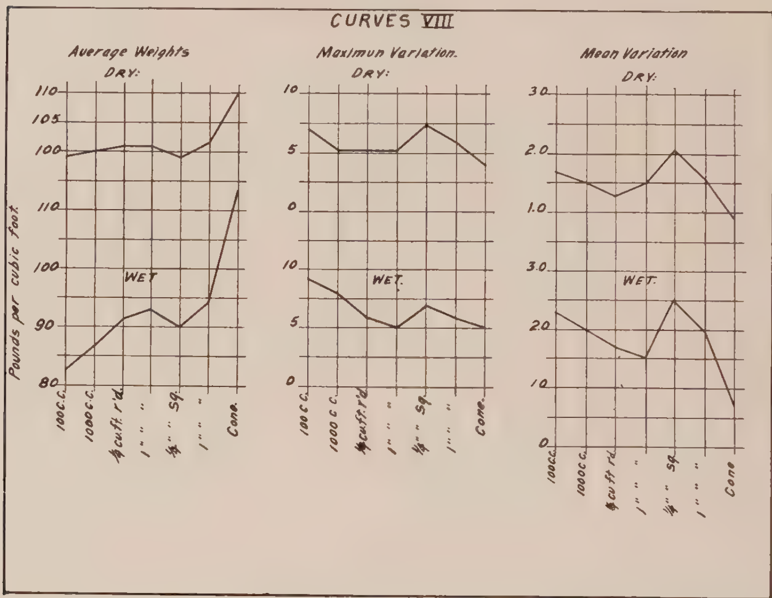

TABLE VIII.—EFFECT OF MEASURE ON AVERAGE WEIGHT, AVERAGE VARIATION FROM MEAN WEIGHT, AND DIFFERENCE BETWEEN HIGHEST AND LOWEST WEIGHTS OF ALL LABORATORIES, FOR ALL RESULTS.

Measure.	100 c.c.	1000 c.c.	¼ cu. ft. rd.	1 cu. ft. rd.	¼ cu. ft. sq.	1 cu. ft. sq.	Cone.
Average weight	91.1	93.3	96.0	96.9	94.8	97.4	111.64
Average maximum variation	8.05	6.88	5.83	5.18	7.10	6.05	4.12
Average variation	2.01	1.74	1.47	1.50	2.25	1.92	0.82

The curves show the effect of the size and shape of the measure used, upon the average weight per cu. ft. the average maximum variation, and the average variation from the mean.

In the matter of average weight it should be noted that the largest measures give the highest weights, and also that the cone method gives weights which are greatly in excess of those obtained with any other measure.

As regards maximum variation the smaller measures give the greatest

range in results, and also that cubical measures give slightly greater range in results than cylindrical measures of equal capacity. The cone method shows the least difference between highest and lowest weights.

The average variation from the mean is least in the case of the larger cylindrical measures and is practically the same for both the ¼ and 1 cu. ft. sizes. The exception is again the cone method which shows much lower average variations from the mean than any other size of measure.

CONCLUSION.

It is evident from even a casual study of curves VII and VIII that with wet sand a much greater range in average weight, average maximum variation and mean variation is obtained than is the case with dry sand. In other words, the dry sand curves are all flatter than the wet sand curves.

Because of this greater range in values with the wet sand, in the discussion below only the results obtained with dry sand are considered. It is to be noted from curves VII at the left for dry sand that the lowest weights are obtained with methods A, F, and G, while the highest weights are obtained by weights C, D, and the cone. Methods B and E give intermediate weights. If, therefore, it were desired to use the method giving the highest weight per cubic foot, the cone method would be first choice with D or C second choice, whereas if low weights are considered desirable, the choice would fall on F, G or A.

Considering now the dry sand curve in the middle of the page, showing average maximum variation, it is very apparent that the cone method is subject to the least total variation with methods D and G as second choice. The largest variations are given by methods A, B and E, which do not differ greatly from each other in this respect.

Passing to the more important function of mean variation shown in the right hand curve, we note that again the cone method shows the lowest figure with method F as second and method D as third choice.

Considering now the size and shape of the measure without regard to the method of filling, we note from an examination of the curve on the left of curve VIII that the weight obtained is slightly higher with the larger round measures than with the smaller round ones, while the cubical measures of ¼ cu. ft. size gives a somewhat lower result, probably due to failure of sand to pack as closely into corners, whereas in the large cubical measure the greater volume offsets this influence of the corners. These differences are, however, minor and may not strongly influence the choice of size, except that one size and shape only should ultimately be adopted as standard. The cone measure, due probably chiefly to the method of filling and compacting gives very much higher weights, so high in fact as to bear little relation to weight per cubic foot of the sand in any condition which would be found in practice.

Passing to average maximum variation shown for dry sand in the middle of the sheet, it is noted that again there is but little to choose between the different sizes of measures, although slightly more variation is shown by the 100 c.c. and the ¼ cu. ft. cubical measure than by the others.

The mean variation curve at the right of the page indicates again that size of measure alone has but small effect on uniformity of results, although, size for size, the round measures are more concordant than the square measures.

Before a definite choice can be made of both method and container, it will be desirable to conduct further investigations to ascertain the degree of compactness of sand in the various conditions in which it exists on the job and in concrete and mortar. Such investigations are in progress and a choice of method and measure will be recommended for adoption when more complete data is gathered.

Sanford E. Thompson,
Chairman.

DISCUSSION.

Mr. D. A. Abrams.—We have been carrying out some investigations of this kind in our laboratory in the Lewis Institute, in Chicago. I was aware of Mr. Chapman's studies, but we did not participate in those investigations; we rather worked out a method for our own use without any intention of recommending it for a standard. This consists of placing the aggregate in a cylindrical measure which has a diameter equal to its height. We use a cast-iron measuring machine on each side, and the aggregate then is simply puddled into the measure by means of a light bar, a slight deviation from the method Mr. Chapman mentioned. This method seems to us to have three distinct advantages. In the first place it is simple and requires no particular apparatus; it conforms exactly to the methods we use in making our own concrete test pieces, it gives us values which are about the same as those found in the actual concrete. **Mr. Abrams.**

REPORT OF THE COMMITTEE ON REINFORCED-CONCRETE CHIMNEYS.

The Committee on Reinforced-Concrete Chimneys has adopted the following comprehensive program of the work which it intends carrying out:

FOUNDATIONS.

- Character of soil.
- Methods of construction, reinforcing, etc.
- How prepared for chimney.

TYPES OF CHIMNEY.

- Cylindrical.

CONSTRUCTION.

- Tapering.
- Single shell (without inner lining).
- Double shell (with inner lining).
- Height required for inner lining.
- Space between inner and outer shell.

DESIGN.

- Design of chimney to resist various forces to which it is subjected.
- Flue opening.
- Collar.
- Types of reinforcement (bars plain or deformed-fabric).
- Location and spacing of reinforcement.
- Splices in reinforcing.

CONCRETE.

- Proportions and mix.
- Suitable aggregate (sand, stone, gravel).
- Consistency of mix (wet or dry).

CONSTRUCTION.

- Forms and methods of moving same.
- Mixing and placing concrete.
- Continuous or intermittent work.
- Placing reinforcement accurately.
- Elapsed time after construction until after chimney is put in service.
- Advisability of constructing chimneys in freezing weather and precautions necessary.

STUDY OF EXISTING CHIMNEYS.

- Effect of heat on concrete.
- Temperature and character of gases in chimneys.
- Chimneys which have resisted unusual conditions, such as hurricanes or earthquakes.

FAILURES—DUE TO.

Design.

Methods of construction—wet or dry mix—continuous or non-continuous work.

Concrete—proportions and aggregate.

Lack of bond between reinforcement and concrete.

Type of reinforcement.

Effect of temperature or gases.

Defective foundations.

Unusual conditions such as hurricanes or earthquakes.

How failures occurred with preliminary indications of weakness.

Percentage of failure; concrete and other types of chimneys.

The various subject-matters of this outline have been assigned to different members of the committee, who will give them special consideration.

During the past year considerable work has been done by individual members of the committee, but this work has not been passed upon by the general committee. However, we will present to the convention papers* dealing with chimney construction which have been prepared by the members of the committee. These are not presented as recommendation but for the purpose of opening a discussion and getting the views of the members of the Institute.

HARRISON W. LATTA,
Chairman.

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* These papers follow this committee report.

COMMENTS ON REINFORCED-CONCRETE FOOTINGS FOR CHIMNEYS.

By Louis R. Cobb.*

It is not the purpose of this paper to attempt to determine and state the best method of calculation and construction for reinforced-concrete foundations for chimneys, either reinforced concrete or brick, but rather to call attention to some of the more common defects in such designs, with the hope that those engaged almost exclusively in this department of engineering may give us some rational and consistent method of design.

Many chimneys are erected every year in connection with our work, the design of which we make a practice of examining in our office. These designs originate in many offices and the lack of any common basis of design is very apparent to anyone comparing the various types. These differences exist not only in designs from the several offices, but also in designs from the same company, and vary from a round stack with a square base, to a square stack with a round base.

Chimney foundations are, probably, subjected to more overloading than any other class of foundations. This condition becomes especially true in many cases of alterations and extensions of existing plants. Chimney foundations generally extend quite a distance beyond the shaft and seem to possess a peculiar and troublesome faculty of always being just where a heavy wall or column of the new portion should be located. Frequently the only answer is "The chimney foundation is large and will carry the new load," and on the new load goes. This leaves one of two conditions, neither of which can be considered good practice. Either the steel, concrete or soil, or all three, in the original foundation is overstressed and liable to failure, or the foundation was improperly designed for the original loading.

Conservative values should be used for the bearing power of the soil because of the peculiar vibratory conditions in chimneys.

The wind load is a very important factor, especially in the case of high chimneys, and should be carefully and thoroughly studied. The wind loads are never gradually applied nor of uniform force, but consist, rather, of a series of shocks or blows suddenly applied and are therefore more severe upon the structure than would be loads of uniform intensity gradually applied. The wind must also be considered as acting in any direction and consequently subjecting every point in the perimeter to a maximum loading.

Theoretically the circular foundation is the most favorable shape for meeting these requirements. In actual operation in the field, however, the difficulty in making circular forms and the number of rods of unequal lengths required may be justly urged against its use. The octagonal shape requires

* Westinghouse Church Kerr & Co., member of Committee on Reinforced-Concrete Chimneys.

only straight formwork and the rods are arranged in bands having rods of equal length and, as the mathematics are very close to those of the circular form, it is very efficient. The square foundation is very uneconomical, as the wind should be, though frequently it is not, considered acting in the direction of the diagonals as well as perpendicularly to the sides. This produces long overhangs and high soil pressures at the corners. This high pressure decreases rapidly toward the center and undoubtedly is distributed, more or less, through the adjacent portions of the foundation and may, therefore, be permitted to exceed somewhat the allowable soil pressure.

Frequently, in plans received at our office, the reinforcing steel is improperly placed. One case is recalled in which one-half of the steel was placed in two planes near the lower face, the other half, also in two planes, being very near the center of the foundation. As the intensity of a stress varies directly as its distance from the neutral axis this steel, so near the neutral axis, could be of little aid in taking tension without greatly overstressing the more distant rods. In fact, it might be more harmful than beneficial to the foundation by tending to cause a cleavage plane near the plane of maximum horizontal shear. Almost invariably all the steel is placed near the lower face of the foundation and generally this distribution is satisfactory. An exception to this, however, occurs when a chimney has a large vertical flue opening and a small foundation. The outer part of the foundation, extending beyond the outer wall of the chimney is treated as a cantilever and reinforced as such, the reinforcing being continued under the flue opening and extending to the opposite side of the footing and acting as cantilever reinforcement in each case.

This method is satisfactory provided the radius of the flue is less, or at least not much greater, than the projection of the foundation beyond the wall of the chimney. The unit loads on the interior portions of the foundations are less, due to wind moment action, than those on the outer portions, consequently the interior loaded length may be greater and still allow the two sections to act as balanced cantilevers. If, however, the radius of the flue is much greater than the overhang of the footing, the central portion will have more load and also a greater moment and will therefore require more steel than the outer cantilever. Under these conditions two systems of reinforcement, top and bottom, are preferable. The rods of the stop system should extend sufficiently beyond the line of the flue opening to obtain bearing under the walls of the chimney. The rods of the bottom system should also extend far enough beyond the line of the flue opening to secure proper anchorage. In reinforced-concrete chimneys each and every rod of the vertical reinforcement must be securely anchored in the foundation. If not, it is useless in the endeavor to make the foundation and superstructure a monolith, as they should be.

Some, at least, of the defects which develop in chimneys after their erection may be ascribed to faulty foundations, and, regardless of how perfect the superstructure may be, defects in the foundations will eventually cause trouble in the completed structure. Defective foundations are almost invariably caused by unwise assumptions and incorrect statics, both of which conditions can easily and quickly be avoided. The difficulties to be surmounted

in the erection of the stack itself, such as pour lines, inequality in the amount of shrinkage and time of setting, expansion and contraction induced by temperature changes, the unequal expansion of the stack walls caused by the difference in temperatures in the outside and inside faces of the walls at the same time, are much more involved and indeterminate, and they should not be further complicated by the use of improperly designed foundations.

If the various organizations chiefly, if not entirely, engaged in chimney construction would, either individually or collectively, adopt a rational and uniform method of designing foundations for chimneys, considerable saving in material and labor in the field and time in the various offices would follow.

CHIMNEY DESIGN.

By Charles P. Woodworth.*

In designing the chimney, the first necessity is to know the capacity for which this chimney is required and whether it be for stills, furnaces, smelting purposes or boilers. If the chimney is to be used in connection with stills, open-hearth furnaces or such high temperature purposes, it is desirable to line the chimney to the top with a good grade of fire-brick, allowing an air space between the inner fire-brick wall and the outer shaft of the chimney, so that the chimney may vibrate due to varying wind-loads and the fire-brick lining expand and contract without doing damage to either.

For ordinary boiler purposes, the lining should be constructed about one-third of the height of the chimney and may be constructed of reinforced concrete, hard burned common brick or fire-brick. Fire-brick, however, is not necessary, as the temperature from boilers scarcely or never exceeds 600° F. in the stack.

If constructed of concrete, there should be a 4-in. air space between the inner and outer wall and the lining should be reinforced both vertically and horizontally to take up all the temperature stresses. The lining should be 4 in. thick. There should be 12 or 14 vertical bars, $\frac{1}{2}$ in. in diameter and ring steel consisting of $\frac{1}{2}$-in. round bars spaced 14-in. centers. If constructed of common brick or fire-brick, the height of a single section of lining should not exceed 60 ft., that is, if the lining were 100 ft. high it should have a 9-in. wall to a height of 40 ft.; the remaining 60-ft. section could be a $4\frac{1}{2}$-in. wall or one thickness of brick. Where a brick lining is used, the air space at the bottom varies according to the height of the chimney. The brick lining is built straight, uniform diameter and the same dimensions at the inside diameter at the top of the stack, which usually gives about a 4-in. air space at the top of the lining and an air space of 1 or 2 ft., at the bottom of the shaft.

For all diameters below 8 ft. at the top, a 4-in. shell thickness at the top is used.† Above this, a 6-in. shell is used unless it is a chimney of more than 400 ft. in height, or an unusual diameter which is seldom or never encountered. The reason for increasing the shell thickness at the top with the diameter is on account of the tendency to flatten due to wind pressure. The increase in shell thickness, for a 1.72 taper is $\frac{3}{16}$ in. increase in shell thickness per section of $4\frac{1}{2}$ ft. of chimney.

The outside diameter increases from the top down $1\frac{1}{2}$ in. per section of $4\frac{1}{2}$ ft. or 1 ft. on a side for every 72 ft. in height, or 2 ft. in diameter for every 72 ft. in height.

The average compression in the concrete of a chimney at the base 175 ft. in height and below, rarely if ever exceeds 350 lb. per sq. in. Above this

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* Weber Chimney Co., Chicago, member of Reinforced-Concrete Chimney Committee.

† From 8 to 20 ft. in diameter a 5-in. shell at the top is used.

height the compression will be 400 to 450 lb. The maximum compression, 450 lb., is only encountered when the design is one of narrow diameter. This compression is the maximum figure with the wind blowing 100 miles per hr., which equals 25 lb. per sq. ft. of the projected area.

The tension in the steel never exceeds 16,000 lb. per sq. in. under the same wind condition. The most generally recognized authority on chimney design is that formula published by Turneaure & Maurer.

The area of the smoke opening is 1.2 times the area of the chimney at the top, or, in other words, to the area of the chimney at the top should be added 20 per cent to determine the size of the smoke opening, thus allowing for the frictional losses of the flue gases through the breeching into the stack and also for the turn into the stack.

The width of the opening should not be greater than two-thirds of the diameter at the top, the area being made up by the height of the opening. The reason for eliminating the width of the opening is to prevent weakening the structure at this point, while the height of the opening does not seriously affect the stability of the chimney. Around the opening, the wall should be thickened so as to provide additional area for the necessary extra steel reinforcement that should be placed on both sides and above and below the opening. The additional steel should be added to be equal to that which has been removed by the making of the opening.

Where circumstances require a larger opening than two-thirds the diameter, or where two openings are required, where the sum of the two widths exceed the diameter at the top buttresses must be built into the chimney wall as a part of the wall, thus strengthening the structure at this point, but if the sum of the two openings do not exceed two-thirds of the inside diameter of the chimney at the top and the openings are opposite, there is no buttress required, unless it is an exceedingly high chimney where two openings are to be provided, the openings to be opposite. The sum of the widths of these two openings may be equal to the inside diameter at the top without buttressing. Over this they must be buttressed.

In designing the foundation the first thing to consider is the soil, and what load it will sustain. If it has a very poor carrying capacity the spread of the foundation will necessarily be increased so as to reduce the pressure per square foot to the carrying capacity of the soil, but if this necessitates an exceedingly large spread it is not practical and therefore a pile foundation should be provided to carry the chimney. In case that the bearing for the chimney is rock or hardpan the spread of the foundation may be decreased over the standard foundation which is usually designed for a pressure of 4000 lb. per sq. ft., up to 150 ft. in height of chimney. From 150 ft. to 175 ft. in height, the usual design is for 4500 lb. per sq. ft. and above this height, owing to the height of the stack, the pressure would vary up to 6000 lb. per sq. ft., but it rarely ever exceeds this pressure.

The foundation is so designed that in any event the overturning moment falls within the middle third. The thickness of the foundation varies from 2 ft. 6 in. thick through the center on a chimney 65 ft. high to 6 ft. 6 in. thick on a chimney 300 ft. high. A higher chimney would be a special design.

The reinforcement of the foundation is made up of two nets of steel. One diagonal net is placed in the bottom of the foundation, 3 to 4 in. above the bottom of the concrete and is designed to take care of the stresses when the wind is blowing diagonally to the foundation. The rectangular net is placed 2 in. above the diagonal net and the bars are laid parallel to the side of the foundation to take up all bending stresses caused by wind pressure.

The spacing of the steel in the foundation depends entirely upon the size of the foundation from the height of the chimney. It is usually stressed to 16,000 lb. per sq. in. For example, the spacing on a 150-ft. chimney with 8-ft. diameter would have the rectangular net spaced at 14-in. centers and the diagonals at 7-in. centers. The vertical steel from the shaft of the chimney extends down into the foundation and is anchored beneath the horizontal steel in the foundation, thus providing a perfect anchorage for the chimney.

The mixture in the foundation should be a mixture of 1 : 3 : 5 or 1 : 3 : 6 using not over 2-in. stone. The mixture for the shaft of the chimney should be 1 : 2 : 4 using not over 1-in. gravel or stone. The horizontal steel in the shaft of the chimney, which is designed to take up all shearing stresses caused by the wind pressure and all temperature stresses, should consist of $\frac{1}{2}$-in. round bars wound spirally at 14-in. centers around the vertical steel. The ends of the horizontal reinforcement should overlap at least 16 in. It may consist of American Steel & Wire Company's woven wire mesh No. 23. Around all openings and in the top of the chimney, extra horizontal reinforcement should be provided.

All concrete should be a wet mixture.

The greatest bending moment will be found at the base of the shaft and therefore the greatest number of square inches of steel is required in this section. The size of the bar depends upon the desired spacing. Naturally, as the bending moment decreases towards the top of the chimney the steel decreases in the same ratio. In the top section of the chimney theoretically no steel would be required but 12 vertical bars are usually used in the top 20 ft., these bars being $\frac{1}{2}$ in. round evenly spaced. The reason for their use is simply to provide an additional factor of safety and to prevent the possibility of cracks developing.

DISCUSSION.

Mr. Lowell. MR. JOHN W. LOWELL.—In a recent investigation of chimney design I had one chimney under examination in which there were a large number of cracks which went vertically practically the whole height of the chimney. Cracks were open in some places as much as ½ in., and near the top of the chimney it seemed as though the smoke was coming out through the cracks. I took a pick and dug into one of the chimney walls 6 in. but found no reinforcement. I knew that the chimney had been reinforced. Then I went to several chimney builders and learned that most of them were putting the same amount of reinforcement regardless of use, size or thickness of wall.

My conclusions were that practically no benefit would be derived from reinforcement near the inner surface of the wall. Reinforcement to be of any use should be near the outer surface, and the thinner the chimney wall the less apt it is to crack. It seems to me that with the temporary expansion caused by a large difference in heat between the inner part of the chimney and the outer surface, there will be a tension and compression stress set up in the concrete. The inner face will be in compression and the outer in tension. Somewhere between there will be a neutral axis, you might say. Now the steel should be near the outer face or in the tensile portion, otherwise the concrete being stronger in compression than in tension, will crack.

I do not believe that it is economical to put enough steel in a chimney wall to keep it from cracking. If this were the case, the stress in the steel must be kept possibly at 6000 lb. per sq. in. or less, but we know that with steel stressed to from 12,000 to 16,000 lb. the cracks would be very small. If the tension steel is distributed sufficiently, that is with small distances between the steel units, the cracks will be spaced over the whole surface and not follow any particular line or point of weakness. If the cracks are spread, they naturally will be small and therefore will not be detrimental to the life of the chimney.

In Turneaure and Maurer's "Reinforced Concrete" there are some theories in reference to temperature steel. I elaborated on them so that, by a set of curves I made, it would be possible to determine the stresses in the temperature steel for any size chimney in any thickness of wall. Naturally with the wall being very thick, the tendency of elongation of the outer surface to the inner surface will be considerable and the stress in the steel cannot be determined from the elongation of the steel; therefore, the thinner the wall, the less the elongation and the smaller the tension will be. That is my reason for believing that a chimney wall should be as thin as possible; so thin that tensile stresses in the horizontal cross-section will be obtained even from wind loads.

Mr. Lord. MR. A. R. LORD.—It is my opinion that in a chimney the temperature reinforcement should be formed of welded wire with a reasonably fine mesh so as to distribute the cracking under ordinary stresses of 8000 to 10,000 lb. and preserve both the appearance and life of the chimney.

Mr. L. R. Cobb.—I would like to ask Mr. Lowell what average difference he found in temperature between the inside and outside face of the stack itself. **Mr. Cobb.**

Mr. Lowell.—My assumption was an average of the three units—the air outside the wall, the air space between the walls and the air in gases inside. There was a difference between the inner and outer faces of concrete of, I think, 125° to 150° F. **Mr. Lowell.**

Prof. F. R. McMillan.—I once measured the temperature in a 6-in. chimney, 2 in. from the face. The temperature of the outer air was 30° F. and the temperature inside the chimney, 2 in. from the outer air was 130° F. I did not go any further in. **Prof. McMillan.**

REPORT OF THE COMMITTEE ON NOMENCLATURE.

The recent and extensive development of reinforced-concrete construction has introduced a large number of new terms into the vocabulary of the engineering profession. These terms appear in reports, plans, specifications, periodical articles and textbooks. Since there is no definite and general understanding as to the meaning of these terms, confusion, ambiguities and misunderstandings have arisen which have led to disputes and litigations. Recent law-suits on "the belt course" and on "flat-slab construction" are matters of common knowledge.

Hence, the American Concrete Institute has deemed it a matter of great importance to present to its members, and to the engineering profession in general through cooperation with other national engineering societies, a list of suggested definitions for terms current in reinforced-concrete design and construction. To this end, a Committee on Nomenclature, consisting of five members and with Mr. Frank C. Wight, chairman, was appointed about five years ago. This committee presented a report at the Eleventh and Twelfth Annual Conventions of the Institute. On Oct. 6, 1916, President L. C. Wason appointed a new committee with Professor A. B. McDaniel as chairman, to continue the work of the original committee.

The present Committee on Nomenclature has deemed it advisable to present a preliminary list of words and definitions which is not intended to be exhaustive or complete but simply suggestive. The following list includes the well understood terms included in the reports of the first committee, but comprises largely those words which are not to be found in any English dictionary. Of the few words which are defined in one or two of the larger dictionaries, the academic definitions are modified to accord with practice and usage.

The committee recognizes the magnitude and difficulty of its task and requests the active cooperation of the members of the Institute in frank criticism of the definitions here proposed. Contributions of additional words and definitions are also desired.

The following list of words and definitions is respectfully submitted: All paragraphs marked with an asterisk are taken from the report of the committee presented at the Twelfth Annual Convention of the Institute.

I. Aggregate.

*The inert material which in combination with cement and water comprises the essential ingredients of concrete.

*1. *Fine Aggregate.*—Natural sand or stone screenings passing when dry a screen having holes $\frac{1}{4}$ in. in diameter.

*2. *Sand.*—The finely divided material resulting from the natural disintegration of rock and graded as defined under "Fine Aggregate."

*3. *Stone Screenings.*—The finely divided product formed by crushing a natural rock and of a size as defined under "Fine Aggregate."

*4. *Coarse Aggregate.*—Inert material which is retained on a screen having holes ¼ in. in diameter and which is incorporated in the concrete in the mixing. The upper limit of its size depends on various conditions; in general, anything above 3 in. is known as a plum, and is added to concrete after mixing and during placing, thus forming rubble or cyclopean concrete.

*5. *Gravel.*—Rounded rock particles graded from fine to coarse and occurring together with loam, clay or other earthy substances in a natural bed or bank and meeting the above requirements for "Coarse Aggregate."

*6. *Crushed Stone.*—The product resulting from crushing natural rock and meeting the above requirements for "Coarse Aggregate."

*7. *Crushed Slag.*—Air-cooled basic blast furnace slag meeting the above requirements for "Coarse Aggregate."

*8. *Cinder.*—The hard waste product of the combustion of anthracite coal.

*9. *Plums.*—Stones of large size added to concrete after mixing and during placing.

*10. *Bank-Run Gravel.*—The normal product of a gravel bank.

*11. *Run-of-Crusher.*—The unscreened output of the stone crusher.

II. CEMENT.

*12. *Portland Cement*—The finely pulverized product resulting from a calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials and to which no addition greater than 3 per cent has been added subsequent to calcination.

*13. *Natural Cement.*—The finely pulverized product resulting from a calcination of an argillaceous limestone at a temperature sufficient only to drive off the carbonic acid gas.

*14. *Sand Cement.*—The finely pulverized product of intimate mixture in varying proportions (generally one-half of each) of silicious sand or other silicious material and portland cement.

*15. *Puzzolan Cement.*—The finely pulverized product of a mechanical mixture of volcanic ashes or basic blast furnace slag with powdered slaked lime. When slag is used this is sometimes known as slag-cement. (There is a cement known in Germany as Eisen-Portland cement, which in England is frequently referred to as slag cement. This is formed by adding to the ground portland cement clinker prepared as a true portland cement, up to 30 per cent of pulverized slag. This should be distinguished from slag cement, as defined above.)

III. CONCRETE.

*An artificial stone formed by the mixture of hydraulic cement with water and an aggregate composed of hard inert particles of various sizes.

*16. *Precast Concrete.*—Concrete which is cast into forms and then hoisted and set in place. (See also "Unit Construction.")

*17. *Reinforced Concrete.*—Concrete in which metal (generally steel) has been embedded in proportionately small sections in such a manner that the metal and the concrete assist each other in taking stress.

*18. *Rubble or Cyclopean Concrete.*—Concrete in which large stones are embedded after mixing and during placing.

19. *Dry Mix.*—Concrete mixed with just enough water to bind the material together.

20. *Wet Mix.*—Concrete mixed with sufficient water to quake when deposited in forms.

IV. Construction.

21. *Book Tile.*—A terra cotta tile designed to be held in place by dovetailing into the tiles adjacent to it and deriving its name from its resemblance to a book.

22. *Deadman.*—A heavy timber embedded in the ground to serve as an anchorage.

23. *Routing Clerk.*—The man who directs the movements of all the materials of which a structure is to be constructed.

24. *Screed.*—A strip of plaster or of wood laid along a wall at intervals to serve as a gage for a mortar or plaster surface.

25. *Sleeper.*—A strip of wood placed at intervals along a concrete slab to serve as a nailing strip for the wooden flooring; often called a screed.

26. *Sleeve.*—A tube or box of wood or metal placed on the forms of a concrete floor in order to provide a hole for the passage of a pipe or wire.

27. *Split Furring.*—A terra cotta tile occupying a space about 2 in. thick and usually formed by splitting a 4-in. hollow tile in half.

28. *Spline.*—A tongue of wood placed in two grooves of adjoining plank in order to make a tight joint between them.

29. *Splined Plank.*—Plank grooved on both edges in order to receive tongues of wood. See "Spline."

30. *Spouting Concrete* (Verb).—To transport concrete from the place of mixing, through inclined troughs or tubes, to the forms.

31. *Unit Construction.*—A system of reinforced-concrete construction in which all the members are cast independently and later erected into place by derricks and grouted into position.

V. Engineering Design and Theory.

32. *Ambursen Dam.*—A reinforced dam consisting of a sloping deck supported by buttresses so arranged that the thrust of the weight of the water on the sloping deck is transmitted by the buttresses to the footing so that the water adds to the stability of the structure.

33. *Belt Course.*—A continuous horizontal member of the outside wall of a building projecting slightly from the elevation and usually molded in order to produce an architectural effect.

34. *Column Head.*—An enlargement of the upper end of a column generally used in connection with flat-slab or mushroom floors.

35. *Column Capital.*—See "Column Head."

36. *Continuous Action.*—The working together of two or more flexural members (columns, beams or slabs) so as to resist to an appreciable degree the stresses occasioned by the application of load to one or more of the members.

37. *Curtain Wall.*—An exterior wall of concrete, brick, tile, or other material, which rests upon the concrete floor construction and does not itse form a supporting member of the building.

38. *Drop Panel.*—The projecting section of a flat-slab floor over the column head. Drop panels are generally square or oblong and usually are employed for structural rather than ornamental purposes.

39. *Flat-Slab Floor.*—A reinforced-concrete floor so constructed that no beams appear beneath the lower surface.

40. *Hollow Dam.*—A dam of reinforced concrete consisting of a sloping deck and a sloping front or apron resting on buttresses and on a footing. (See "Ambursen Dam.")

41. *Mushroom Floor.*—(See "Flat-Slab Floor.") A special form of flat-slab floor. The term "mushroom" is much used interchangeably with the term "flat-slab."

42. *Mushroom Head.*—The enlarged head or capital of a concrete column which is used to support the flat-slab or mushroom floor of a reinforced concrete structure.

43. *Negative Reinforcement.*—Steel reinforcement inserted in a reinforced concrete structure to take up negative bending moments.

44. *Punching Shear.*—The tendency of a concrete column or other vertical member to shear or punch through the footing upon which it rests.

45. *Wall Beam.*—A beam at the edge of the floor slab and which shows on the outside face of the building.

46. *Weep Hole.*—A hole in a wall to provide drainage for the wall backing.

47. *Well (Elevator).*—A vertical compartment or chamber in a building to receive a passenger or freight elevator.

VI. FINISH.

48. *Carborundum Rubbing.*—Rubbing with a carborundum stone to smooth a surface using a little cement and water.

49. *Cement Wash.*—An application of a mixture of cement and water generally applied with a brush to the surfaces of concrete work, to reduce the permeability or to give a uniform color and appearance to the concrete work.

50. *Composition Flooring.*—A floor surface formed of various chemicals and a filler mixed with water and laid in a plastic condition, troweled smooth and then allowed to set hard.

51. *Granolithic.*—A floor surface formed by a mixture of cement and very fine crushed stone and sometimes an addition of sand, it being troweled smooth while wet and allowed to harden.

52. *Gunite or Guncrete.*—A trade designation for a mortar composed of sand and cement placed by the cement gun.

53. *Plaster and Plastering.*—A mixture of cement and sand or lime and sand, usually with some hair added, applied in successive coats to vertical surfaces and soffits.

54. *Tooling.*—The finishing of concrete surfaces with a special hand or power tool producing a surface which will show the lines of the tool.

VII. FORMS.

A construction to receive wet concrete, generally removed after the concrete is set. (Also known as formwork, falsework or centering.)

55. *Beam Bottom Form.*—A series of boards fastened together with cleats on one side and used for forming the bottom of a beam.

56. *Beam Side Form.*—A series of boards fastened together with cleats on one side and used for forming the side of a beam.

57. *Form Brace.*—A timber placed diagonally between the posts in floor formwork to stiffen the supports.

58. *Form Cleat.*—A strip of wood used to hold together a series of boards forming a panel for use as a wall or floor form.

59. *Column Form Clamp.*—An arrangement of wood, steel or iron members designed to hold forms in position to resist the pressure of wet concrete against their sides.

60. *Column Side Form.*—A series of boards fastened together with cleats or yokes and used for forming the side of a column.

61. *Column Form Yoke.*—A wood or iron member designed to resist the bursting tendency of wood column forms when full of wet concrete.

62. *Form Girt.*—A longitudinal timber laid cross-wise under the joists and used as a girder to support the floor formwork.

63. *Form Jack.*—A vertical support for the formwork of beams and provided with a cross-piece at the top and two short diagonal braces.

64. *Form Joist.*—A longitudinal timber placed under the panels of floor formwork.

65. *Form Ledger.*—A horizontal timber used to brace the posts in floor formwork.

66. *Form Mud Sill.*—A part of the formwork of the floors consisting of members of timber laid upon the surface of the rough ground and wedged up in order to provide a level bearing for the posts of the floor formwork.

67. *Form Panel.*—A group of boards fastened together with cleats on one side and used for forming the lower surface of floor slabs and for walk surfaces.

68. *Form Post.*—A vertical timber used to support the formwork of floors.

69. *Form Shore.*—A temporary vertical timber used to support the weight of a floor after stripping forms until thoroughly set.

70. *Form Re-Stud.*—See "Form Shore."

71. *Form Ribband.*—A horizontal timber used to hold the bottoms of beam sides in place.

72. *Angle Fillet.*—A triangular strip of wood which is placed in the angle of rectangular column or beam forms in order to produce a chamfered edge in the concrete.

73. *Handle Nut.*—A nut with a handle forged on same, and used in form-work and other temporary construction work to obviate the necessity for using wrenches.

74. *Insert.*—A socket, usually of cast or malleable iron, either slotted or tapped to receive bolts for attaching shafting, sprinker pipes or other articles to a concrete surface. Inserts are usually placed upon the forms before the concrete is poured and remain embedded in the concrete after the forms are stripped.

75. *Wall Bolt.*—An iron bolt threaded at both ends used in formwork to bolt the forms of the two sides of a wall together, and also used in column forms in a similar manner.

76. *Wing Nut.*—A nut with two opposite projecting wings so arranged as to obviate the use of wrenches and used in formwork and other temporary construction work.

VIII. REINFORCEMENT.

*The metal (generally steel) embedded in concrete in proportionately small sections in such a manner that the two materials assist each other in taking stress.

77. *Corrugated Bar.*—A steel bar for reinforcing concrete and which has square or oblong projections on its surface formed by special rolls.

78. *Deformed Bar.*—A steel bar for reinforcing concrete and which has projections on its surface in order to secure a mechanical bond between the concrete and the steel. These projections are formed either by passing the bar through specially shaped rolls or by twisting the bar.

79. *Expanded Metal.*—A form of concrete reinforcement made of sheet steel which has been slit and pulled out to form a diamond mesh.

*80. *Hard Steel.*—Steel with a minimum ultimate tensile strength of 80,000 lb. per sq. in. and a minimum yield point of 50,000 lb. per sq. in.

81. *Hickey Bar*—A bar wired to the main reinforcement of a floor to hold the bars in the correct position and at the right elevation.

82. *Hooping.*—A steel or wire band placed at regular intervals around the column steel to hold the same in position while the concrete is being poured and often designed to allow higher stresses in the concrete of the column and to resist the hoop tension.

83. *Radial Bar.*—A bar placed in the column of a mushroom floor and bent down and set so that the ends radiate from the center of the column. These bars support the main floor reinforcement and act as "hickey bars."

84. *Spacer.*—A metal bar placed in a concrete floor, wall or column and used to tie the reinforcement together and hold it in its proper position.

85. *Spiral Hooping.*—A bar bent in a spiral form for wrapping the vertical reinforcement in a column and designed to allow higher stresses in the concrete of the column and to resist the bursting tendency in a heavily loaded column.

86. *Stirrup.*—A piece of metal reinforcement placed in a beam in order to resist tensile stresses in the concrete set up by a combination of shear and longitudinal tension.

*87. *Structural Steel.*—Steel with a minimum ultimate tensile strength varying between 55,000 and 70,000 lb. per sq. in. and a minimum yield point of 33,000 lb. per sq. in.

88. *Twisted Bar.*—A steel bar which has been twisted either before or after cooling in order to raise its elastic limit. Bars twisted before cooling are known as hot twisted and those which are twisted after cooling are known as cold twisted.

89. *Wire Fabric.*—A form of concrete reinforcement composed of parallel longitudinal wires tied together at intervals by transverse wires. The intersecting wires are sometimes welded together. (Also known as "Wire Cloth" or "Wire Mesh.")

IX. TOOLS AND PLANT.

90. *Buggy (Concrete).*—A two-wheeled hand cart for the transporting of wet concrete.

91. *Cement Gun.*—A trade name applied to an apparatus used for the placing of mortar under pressure, the characteristics being that the mortar is forced dry to the nozzle, hydration taking place at the nozzle and coincident with the application.

92. *Chute (Concrete).*—An arrangement of troughs or tubes through which wet concrete is transferred from a mixer or hoisting tower to the forms.

93. *Gin Pole.*—A pole or mast to which a block and tackle are attached for the purpose of hoisting materials.

94. *Guy Derrick.*—A derrick the mast of which is secured by guy ropes to "dead men" or to other anchorages.

95. *Hickey.*—A tool about three feet long with hooked ends used while the floors are being concreted to lift the reinforcement bars in the wet concrete so that the concrete can flow around and under them when pouring a floor.

96. *Traveler.*—A temporary structure, usually of steel but sometimes of wood, which is employed to raise into position the structural members of a bridge or other structure and which in itself is so constructed that it can move backward and forward in accordance with the progress of the work.

97. *Tremie.*—A sectional, cylindrical spout used in placing concrete under water.

X. MISCELLANEOUS.

98. *Billet-Steel Bars.*—Bars rolled from new billets.

99. *Rail-Steel Bars.*—Bars rolled from standard section T-rails.

100. *Grout* (noun).—The resulting mixture of cement and water or cement, sand and water, in fluid consistency.

101. *Grout* (verb).—To grout is to fill a cavity or joint with a mixture of cement and sand.

102. *Laitance.*—A scum of finely divided inert particles which rises to the surface of wet concrete during setting.

*103. *Mortar.*—A mixture of sand and cement with plaster, or with lime, wetted and mixed to the consistency of paste. Applied to concrete,

the mixture of cement and fine aggregate, which lies in the voids of the coarse aggregate.

104. *Plaster* (noun).—A mixture of cement and sand, or lime and sand usually with some hair added, applied in successive coats to vertical surfaces and soffits.

105. *Plaster* (verb).—To plaster is to apply to flat surfaces with a trowel a mixture of cement and sand, or lime and sand.

106. *Point* (verb).—To point is to apply a mixture of cement and sand to a joint between the edges of a steel sash and the concrete.

Respectfully submitted,

A. B. McDANIEL, *Chairman*,
LESLIE H. ALLEN,
C. D. GILBERT,
G. A. HOOL,
W. A. SLATER.

REPORT OF COMMITTEE ON SIDEWALKS AND FLOORS.

Your Committee on Sidewalks and Floors desires to submit the following report:

Changes in the personnel of the committee have been made necessary by the resignation of R. J. Wig as a member, and the request of L. R. Ferguson that he be relieved of his duties as chairman on account of press of other work. H. H. Davis, of the Bureau of Standards, has accepted membership on the committee in place of Mr. Wig, and it was upon Mr. Ferguson's suggestion that the present chairman was appointed by the Executive Committee of the Institute in September.

Efforts have been made by correspondence and other means of investigation to develop information on various subjects set forth in the suggested program for a study of sidewalks and floors that was presented at the last convention. Considerable data have already been collected but further investigation is desirable.

The matter of sub-base for sidewalks is worthy of special consideration. A porous sandy or gravelly soil has a good natural drainage, and in such cases the walk might well be laid directly upon the sub-grade and the sub-base eliminated. We have already secured the specifications of one city in the central states that is following similar practice at this time. In heavy soils a cinder or gravel-filled trench beneath the walk may be a detriment rather than an advantage, unless properly drained.

In the construction of concrete floors there seems to be a tendency to mix the top mortar too wet. The use of stiffer mixtures, less troweling of the mortar finish, and the prevention of rapid drying during early hardening by a suitable covering are all factors of vital importance to the obtaining of a durable surface with high resistance to abrasion.

The committee feels that further time is needed in which to secure more information on a number of points before submitting a detailed report on recommended practice, but in presenting this progress report it seems desirable to submit for the consideration of the Institute proposed revised specifications for sidewalks and for floors that can be tentatively accepted for a year, at the end of which period the additional information collected by the committee, and the consideration of these specifications by the Institute as a whole will undoubtedly suggest improvements and additions.

Respectfully submitted,

J. E. Freeman,
Chairman.

PROPOSED REVISED SPECIFICATIONS FOR CONCRETE SIDEWALKS.

These specifications apply to concrete sidewalks in residence and business districts, and cover the preparation of the sub-base, the laying and finishing of the walk and its protection during early hardening.

GENERAL REQUIREMENTS.

Materials.

1. *Cement.*—The cement shall meet the requirements of the current Standard Specifications for Portland Cement of the American Society for Testing Materials.

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the architect or engineer a fifty (50) lb. sample of each of the aggregates proposed for use. These samples shall be tested and if found to pass the requirements of the specifications, similar material shall be considered as acceptable for the work. In no case shall aggregates containing frost or lumps of frozen material be used.

(*a*) Fine Aggregate.—Fine aggregate shall consist of natural sand or screenings from hard, tough, crushed rock or gravel consisting of quartzite grains or other equally hard material, graded from fine to coarse, with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes to the linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch, and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other organic matter nor more than three (3) per cent, by weight, of clay or loam. Field tests may be made by the architect or engineer on fine aggregate as delivered at any time during progress of the work. If there is more than seven (7) per cent of clay or loam, by volume, in one (1) hour's settlement after shaking in an excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement and three (3) parts fine aggregate, by weight, when made into briquettes, shall show a tensile strength, at seven (7) and twenty-eight (28) days, at least equal to the strength of briquettes composed of one (1) part of the same cement and three (3) parts Standard Ottawa sand by weight. The percentage of water used in making the briquettes of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquettes of standard consistency. In other respects all briquettes shall be made in accordance with the methods of testing cement

recommended by the American Society for Testing Materials. (See Am. Conc. Inst. Standard No. 1.)

(*b*) Coarse Aggregate.—Coarse aggregate shall consist of clean, durable, crushed rock or pebbles graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall range from one (1) in. down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

(*c*) No. 1 Aggregate for Wearing Course.—No. 1 aggregate for the wearing course shall consist of clean, hard, tough, crushed rock or pebbles, free from vegetable or other organic matter and shall contain no soft, flat or elongated particles. It shall pass, when dry, a screen having one-half ($\frac{1}{2}$) in. square openings and not more than ten (10) per cent shall pass a screen having four (4) meshes per linear inch.

3. *Mixed Aggregate.*—Crusher-run stone, bank-run gravel or mixtures of fine and coarse aggregate prepared before delivery on the work shall not be used.

4. *Sub-base.*—Only clean, durable material, such as coarse gravel or steam-boiler cinders free from ash or particles of unburned coal shall be used in the sub-base. (Note: Eliminate this clause when sub-base is not required.)

5. *Water.*—Water shall be clean, free from oil, acid, alkali, vegetable or other organic matter.

6. *Color.*—If artificial coloring material is required only those mineral colors shall be used which, in the amount hereinafter specified, will not appreciably impair the strength of the cement.

7. *Reinforcement.*—Reinforcing metal shall meet the requirements of the current Standard Specifications for Steel Reinforcement of the American Society for Testing Materials. It shall be free from excessive rust, scale, paint or coatings of any character which will tend to reduce or destroy the bond. The reinforcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft.

8. *Joint Filler.*—The joint filler shall be a suitable elastic waterproof compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather, or prepared strips of fiber matrix and bitumen as approved by the architect or engineer. The strips shall be one-half ($\frac{1}{2}$) in. in thickness, their width shall at least equal the full thickness of the slab and their length shall at least equal the width of the slab at the joint.

SUB-GRADE.

9. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness. The sub-grade shall be thoroughly tamped until it is brought to a firm, unyielding surface. It shall have a slope toward the street curb of not less than one-half ($\frac{1}{2}$) in. per ft.

When the concrete sidewalk is to be constructed over an old path composed of gravel or cinders, the old path shall be entirely loosened, the material spread for the full width of the sub-grade and compacted as specified.

10. *Deep Fills.*—All fills shall be made in a manner satisfactory to the architect or engineer. The use of muck, quicksand, soft clay, spongy or perishable material is prohibited. The top of all fills shall extend beyond the walk on each side at least one (1) ft., and the sides shall have a slope not greater than one (1) or one and one-half ($1\frac{1}{2}$).

11. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the architect or engineer.

12. *Depth.*—The sub-grade shall be not less than —— (–) in. below the finished surface of the walk.

> NOTE.—Sub-grade to be at least five (5) in. below the finished surface of the walk when sub-base is not required and at least eleven (11) in. below when sub-base is required.

SUB-BASE.

(Omit these sections when sub-base is not required.)

13. *Thickness.*—On the sub-grade shall be spread a material as hereinbefore specified which shall be thoroughly rolled or tamped to a surface at least —— (–) in. below the finished grade of the walk. On fills, the sub-base shall have the same slope as the sides of the fill.

14. *Wetting.*—While compacting the sub-base, the material shall be kept, thoroughly wet and shall be wet when the concrete is deposited, but shall show no pools of water.

FORMS.

15. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

16. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the walk.

17. *Division Plates.*—Suitable metal division plates shall be provided to completely separate adjacent slabs during construction unless otherwise permitted by the architect or engineer.

18. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled or coated with soft soap or whitewash before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

19. *Size of Slabs.*—The slabs or independently divided blocks when not reinforced shall have an area of not more than one hundred (100) sq. ft., and shall not have dimensions greater than ten (10) ft., nor shall the length of any such slab be greater than one and one-half ($1\frac{1}{2}$) times the width. Larger slabs shall be reinforced as hereinafter specified.

20. *Thickness of Walk.*—The thickness of the walk shall be not less than —— (–) in.

NOTE.—Walks for residence districts to be at least five (5) in. thick and for business districts six (6) in. Where walks cross driveways in residence districts, the total thickness shall be increased to six (6) in. with a one (1) in. wearing course.

21. *Joints.*—A one-half (½) in. joint shall be provided at least once every fifty (5) ft. in the length of the walk which shall be filled with suitable material as specified under "Joint Filler." A similar joint shall be provided at each intersection of sidewalk and street curb and at such other points as may be designated by the architect or engineer.

22. *Protection of Edges.*—Where required by the architect or engineer, the edges of the slabs at the joints shall be protected by metal. Unless protected by metal, the upper edges of the slabs shall be rounded to a radius of one-half (½) in. The edges of all slabs abutting a business street which act as curbing must be protected by suitable metal angles or corner-bars as approved by the architect or engineer.

MEASURING AND MIXING.

23. *Measuring.*—The method of measuring the materials for the concrete or mortar, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 lb. net) shall be considered one (1) cu. ft.

24. *Machine Mixing.*—All concrete shall be mixed by machine except when otherwise permitted under special conditions. A batch mixer of any approved type shall be used. The ingredients of the concrete or mortar shall be mixed to the specified consistency, and the mixing shall continue for at least one (1) minute after all materials are in the drum. The drum shall be completely emptied before receiving material for the succeeding batch.

25. *Hand Mixing.*—When it is necessary to mix by hand, the materials shall be mixed dry on a water-tight platform until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous.

26. *Retempering.*—Retempering of mortar or concrete which has partially hardened, that is, remixing with or without additional materials or water, shall not be permitted.

PROTECTION.

27. *Treatment.*—As soon as the finished walk has hardened sufficiently to prevent damage, the surface of the walk shall be sprinkled with clean water or preferably covered with at least one (1) in. of wet sand or earth, and kept wet for at least seven (7) days.

28. *Protection.*—The freshly-finished walk shall be protected from hot sun and drying winds until it can be sprinkled and covered as above specified. The concrete surface must not be damaged or pitted by rain drops, and the contractor shall provide and use when necessary sufficient tarpaulins to com-

pletely cover all sections that have been placed within the preceding twelve (12) hours. The contractor shall erect and maintain suitable barriers to protect the walk from traffic and any section damaged from traffic or other causes, occurring prior to its official acceptance, shall be repaired or replaced by the contractor at his own expense in a manner satisfactory to the architect or engineer. Before the sidewalk is opened to traffic the covering shall be removed and disposed of by the contractor. The walk shall not be opened to traffic until the architect or engineer so directs.

29. *Temperature Below 35° Fahrenheit.*—If at any time during the progress of the work the temperature is, or in the opinion of the architect or engineer will within twenty-four (24) hours drop to 35° Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days.

TWO-COURSE SIDEWALK.

For two-course walks the following will apply in addition to the general requirements:

CONCRETE BASE.

30. *Proportions.*—The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two and one-half ($2\frac{1}{2}$) cu. ft. of fine aggregate and five (5) cu. ft. of coarse aggregate.

31. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

32. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established grade of the walk. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. If dirt, sand or dust collect on the base it shall be removed before the wearing course is applied. Workmen shall not be permitted to walk on the freshly-laid concrete. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

33. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having dimensions greater than ten (10) ft., shall be reinforced with wire fabric or with plain or deformed bars. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal. The reinforcement shall not be less than one (1) in. from the finished surface of the walk.

WEARING COURSE.

34. *Proportions for Mixture No. 1.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement and not more than two (2) cu. ft. of fine aggregate. The minimum thickness shall be three-quarter ($\frac{3}{4}$) in. (Note. Proportions and thickness for residence districts or where traffic is light.)

35. *Proportions for Mixture No. 2.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement, one (1) cu. ft. of fine aggregate and one (1) cu. ft. of "No. 1 Aggregate for Wearing Course." The minimum thickness shall be one (1) in. (Note: Proportions and thickness for business districts or where traffic is heavy.)

36. *Consistency.*—The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

37. *Placing.*—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard. In no case shall more than forty-five (45) minutes elapse between the time the concrete for the base is mixed and the wearing course is placed.

38. *Finishing.*—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel troweled, but excessive working shall be avoided. In no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface to absorb moisture or to hasten the hardening. When division plates are not used, the slab markings shall be made in the wearing course directly over the joints in the base with a tool which will completely separate the wearing course of adjacent slabs. Unless protected by metal the surface edges of all slabs shall be rounded to a radius of one-half ($\frac{1}{2}$) in.

39. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course, and shall be mixed dry with the cement and aggregate until the mixture is of a uniform color. In no case shall the amount of coloring exceed eight (8) per cent of the weight of the cement.

ONE-COURSE SIDEWALK.

For one-course walks the following will apply in addition to the general requirements.

40. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of Portland cement to not more than two (2) cu. ft. of fine aggregate and not more than three (3) cu. ft. of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the coarse aggregate.

A cubic yard of concrete in place shall contain not less than six and eight-tenths (6.8) cu. ft. of cement.

41. *Consistency.*—The materials shall be mixed with sufficient water to produce a concrete which will hold its shape when struck off with a strike-

board. The consistency shall not be such as to cause a separation of the coarse aggregate from the mortar in handling.

42. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete brought to the established grade with a strikeboard. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. Workmen shall not be permitted to walk on the freshly-laid concrete. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

43. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having dimensions greater than ten (10) ft., shall be reinforced with wire fabric or with plain or deformed bars. The reinforcement shall be placed two (2) in. below the finished surface of the walk. The reinforcement shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

44. *Finishing.*—After the concrete has been brought to the established grade by means of a strikeboard, it shall be floated with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel troweled, but excessive working shall be avoided. Unless protected by metal the surface edges of all slabs shall be rounded to a radius of one-half (½) in.

PROPOSED REVISED SPECIFICATIONS FOR CONCRETE FLOORS.

These specifications apply to floors in buildings, whether subjected to moderate or heavy traffic, and cover the laying and finishing of the floor; also its protection during early hardening.

For architects, engineers and others desiring to embody these specifications in their general specifications covering a particular piece of work, the following outline of the paragraphs necessary to meet different conditions will prove convenient:

Floors Laid on Ground.—Moderate or light traffic.

Two-Course.—Paragraphs 1–15 (except 2*c*); 30–47; 49–52.

One-Course.—Paragraphs 1–15 (except 2*c* and *b*); 30–42; 53–57.

Floors Laid on Ground.—Heavy traffic.

Two-Course.—Paragraphs 1–15; 30–46; 48–52.

Reinforced Concrete Floors.—Moderate or light traffic; paragraphs 1–22 (except 2*c*); 24–29. Heavy traffic; Paragraphs 1–23; 25–29.

GENERAL REQUIREMENTS.

MATERIALS.

1. *Cement.*—The cement shall meet the requirements of the current Standard Specifications for Portland Cement adopted by the American Society for Testing Materials. (Am. Conc. Inst. Standard No. 1.)

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the architect or engineer a fifty (50) lb. sample of each of the aggregates proposed for use. These samples shall be tested, and if found to pass the requirements of the specifications, similar material shall be considered as acceptable for the work. In no case shall aggregates containing frost or lumps of frozen material be used.

(*a*) Fine Aggregate.—Fine aggregate shall consist of natural sand or screenings from hard, tough, crushed rock or gravel, consisting of quartzite grains or other equally hard material, graded from fine to coarse, with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes to the linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch; and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other organic matter nor more than three (3) per cent by weight of clay or loam. Field tests may be made by the architect or engineer on fine aggregate as delivered

at any time during progress of the work. If there is more than seven (7) per cent of clay or loam by volume in one (1) hour's settlement after shaking in an excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement and three (3) parts fine aggregate, by weight, when made into briquettes, shall show a tensile strength at seven (7) and twenty-eight (28) days at least equal to the strength of briquettes composed of one (1) part of the same cement and three (3) parts standard Ottawa sand, by weight. The percentage of water used in making the briquettes of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquettes of standard consistency. In other respects all briquettes shall be made in accordance with the methods of testing cement recommended by the American Society for Testing Materials. (See Cement Specifications A. S. T. M.)

(*b*) Coarse Aggregate.—Coarse aggregate shall consist of clean, hard, tough, crushed rock or pebbles graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall range from one (1) in. down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

(*c*) No. 1 Aggregate for Wearing Course.—No. 1 aggregate for the wearing course shall consist of clean, hard, tough, crushed rock or pebbles, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. It shall pass when dry a screen having one-half ($\frac{1}{2}$) in. square openings and not more than ten (10) per cent shall pass a screen having four (4) meshes per linear inch.

3. *Mixed Aggregate.*—Crusher-run stone, bank-run gravel or mixtures of fine and coarse aggregate prepared before delivery on the work shall not be used.

4. *Sub-base.*—Only clean, hard material, such as coarse gravel or steam-boiler cinders, free from ash or particles of unburned coal, shall be used in the sub-base. (NOTE.—Eliminate this clause when sub-base is not required.)

5. *Water.*—Water shall be clean, free from oil, acid, alkali, vegetable or other organic matter.

6. *Color.*—If artificial coloring matter is required, only those mineral colors shall be used which, in the amount hereinafter specified, will not appreciably impair the strength of the cement.

7. *Reinforcement.*—The reinforcing metal shall meet the requirements of the current Standard Specifications for Steel Reinforcement of the American Society for Testing Materials. It shall be free from excessive rust, scale, paint or coatings of any character which will tend to reduce or destroy the bond.

8. *Joint Filler.*—The joint filler shall be a suitable compound that will not become soft and run out in hot weather, nor hard and brittle and chip out in cold weather; or, prepared strips of fiber matrix and bitumen as approved by the architect or engineer. The strips shall be one-half ($\frac{1}{2}$) in. in thickness and their width shall at least equal the full thickness of the slab.

Measuring and Mixing.

9. *Measuring.*—The method of measuring the materials for the concrete or mortar, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 lb. net) shall be considered as one (1) cu. ft.

10. *Machine Mixing.*—All concrete shall be mixed by machine except when the architect or engineer shall otherwise permit under special conditions. A batch mixer of an approved type shall be used. The ingredients of the concrete or mortar shall be mixed to the specified consistency, and the mixing shall continue for at least one (1) minute after all the materials are in the drum. Raw materials shall not be permitted to enter the drum until all the material of the preceding batch has been discharged.

11. *Hand Mixing.*—When it is necessary to mix by hand, the materials shall be mixed dry on a water-tight platform until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous.

12. *Re-tempering.*—Re-tempering of mortar or concrete which has partially hardened, that is, re-mixing with or without additional materials or water, shall not be permitted.

Protection.

13. *Treatment.*—As soon as the finished floor has hardened sufficiently to prevent damage thereby, the floor shall be covered with at least one (1) in. of wet sand, or two (2) in. of sawdust, which shall be kept wet by sprinkling with water for at least ten (10) days.

14. *Protection.*—The freshly-finished floor shall be protected from hot sun and drying winds until it can be sprinkled and covered as above specified. The concrete surface must not be damaged or pitted by raindrops, and the contractor shall provide and use when necessary sufficient tarpaulins to completely cover all sections that have been placed within the preceding twelve (12) hours.

15. *Temperature Below 35° Fahrenheit.*—If at any time during the progress of the work the temperature is, or in the opinion of the architect or engineer will within twenty-four (24) hours drop to 35° Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days.

REINFORCED-CONCRETE FLOORS.

For reinforced-concrete floors the following will apply in addition to the general requirements:

16. *Forms.*—The forms shall be substantial, unyielding and so constructed that the concrete will conform to the designed dimensions and contours, and shall also be tight to prevent the leakage of mortar. The supports for floors shall not be removed in less than ten (10) days after the concrete is placed, and then only with the consent of the architect or engineer in

charge. When freezing weather occurs, the supports shall remain in place an additional time, equal to the time the floor has been exposed to freezing.

17. *Reinforcement.*—Reinforcing metal shall be provided as called for on the plans. It shall be placed as indicated and mechanically held in position so that it will not become disarranged during the depositing of the concrete. Whenever it is necessary to splice tension reinforcement, the character of the splice shall be such as will develop its full strength. Splices at points of maximum stress shall be avoided. Splicing by lapping bars without contact and with space between bars along the over-lap equal to twice the thickness of the bars is preferable to mechanical splices or clamps.

CONCRETE SLAB.

18. *Proportions.*—The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two (2) cu. ft. of fine aggregate and four (4) cu. ft. of coarse aggregate.

19. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that may be readily caused to flow into the forms and about the reinforcement, but which can be conveyed from the mixer to the forms without the separation of the coarse aggregate from the mortar.

20. *Placing.*—The concrete shall be placed in a manner to insure a smooth ceiling, and thoroughly worked around the reinforcement and into the recesses of the forms. Concrete shall be deposited in its full position as soon as possible after mixing and within thirty (30) minutes the water after has been added to the dry materials. It shall be struck off to a surface at least (1) in. below the established grade of the finished surface of the floor. Workmen shall not be permitted to walk in freshly-laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

21. *Joints.*—When it is necessary to make a joint in a floor slab, its location shall be designated by the architect or engineer; joints to be vertical.

WEARING COURSE.

22. *Proportions and Thickness (Mixture No. 1).*—The mortar shall be mixed in the proportions of one (1) sack of portland cement and two (2) cu. ft. of fine aggregate. The minimum thickness shall be three-quarters ($\frac{3}{4}$) in.

23. *Proportions and Thickness (Mixture No. 2).*—The mortar shall be mixed in the proportions of one (1) sack of portland cement, one (1) cu. ft. of fine aggregate and one (1) cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be one (1) in.

24. *Consistency.*—The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

25. *Placing.*—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard.

NOTE.—When placing the wearing course after the concrete slab has hardened, eliminate paragraph 25 and substitute the following:

26. *Preparation of Slab.*—The surface of the slab shall be thoroughly roughened by picking, and swept clean of all dirt and débris.

27. *Placing.*—The slab shall be thoroughly moist but free from pools of water when the grout and mortar for wearing course is placed. A neat cement grout shall be brushed on the surface of the slab, the wearing course immediately applied and brought to the established grade with a strikeboard. Grout and mortar shall be used within forty-five (45) minutes after mixing with water.

28. *Finishing.*—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. In no case shall dry cement or mixture of dry cement and sand be sprinkled on the surface to absorb moisture or to hasten the hardening, but the Bruner method may be used if desired.

29. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

PLAIN CONCRETE FLOORS.

For plain concrete floors the following will apply in addition to the general requirements:

SUB-GRADE.

30. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness. The sub-grade shall be thoroughly tamped until it is brought to a firm, unyielding surface.

31. *Deep Fills.*—All fills shall be made in a manner satisfactory to the architect or engineer. The use of muck, quicksand, soft clay, spongy or perishable material is prohibited.

32. *Drainage.*—When required, a suitable drainage system shall be installed and connected with sewers or other drains indicated by the engineer.

33. *Depth.*—The sub-grade shall be not less than ——— (—) in. below the finished surface of the floor.

> NOTE.—Sub-grade to be five (5) in. below the finished surface of the floor when sub-base is not required, and at least eleven (11) in. below when sub-base is required.

SUB-BASE.

(Omit these sections when sub-base is not required.)

34. *Thickness.*—On the sub-grade shall be spread a material as hereinbefore specified, which shall be thoroughly rolled or tamped to a surface at

least ——— (—) in. below the finished grade of the floor. On fills the sub-base shall extend the full width of the fill.

35. *Wetting.*—While compacting the sub-base, the material shall be kept thoroughly wet, and shall be in that condition when the concrete is deposited.

FORMS.

36. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

37. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the floor.

38. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled or coated with soft soap or whitewash before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

39. *Size of Slabs.*—The slabs or independently-divided blocks when not reinforced shall have an area of not more than one hundred (100) sq. ft., and shall not have dimensions greater than ten (10) ft. Larger slabs shall be reinforced as hereinafter specified.

40. *Thickness of Floor.*—The thickness of the floor shall be not less than five (5) in.

41. *Width and Location of Joints.*—When required by the architect or engineer in charge, a one-half (½) in. space or joint shall be left between the floor and the walls and columns of the building, to be filled with the material before specified under "Joint Filler."

42. *Protection of Edges.*—Where required by the architect or engineer in charge, the edges of the slabs at the joints shall be protected by metal. Unless protected by metal, the upper edges of the slabs shall be rounded to a radius of one-half (½) in.

TWO-COURSE FLOOR.

(Concrete Base.)

43. *Proportions.*—The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two and one-half (2½) cu. ft. of fine aggregate and five (5) cu. ft. of coarse aggregate.

44. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

45. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the

concrete struck off and tamped to a surface the thickness of the wearing course below the established elevation of the floor. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. If dirt, sand or dust collects on the base it shall be removed before the wearing course is applied. Workmen shall not be permitted to walk on the freshly-laid concrete. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

46. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having dimensions greater than ten (10) ft., shall be reinforced with wire fabric, or with plain or deformed bars. The reinforcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

(Wearing Course.)

47. *Proportions for Mixture No. 1.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement, two (2) cu. ft. of fine aggregate. The minimum thickness shall be three-quarters ($\frac{3}{4}$) in.

48. *Proportions for Mixture No. 2.*—The wearing course shall be mixed in the proportions of one (1) sack of Portland cement and one (1) cu. ft. of fine aggregate, and one (1) cu. ft. on No. 1 aggregate for wearing course. The minimum thickness shall be one (1) in.

49. *Consistency.*—The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

50. *Placing.*—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard. In no case shall more than forty-five (45) minutes elapse between the time the concrete for the base is mixed and the wearing course is placed.

51. *Finishing.*—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. In no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface to absorb moisture or to hasten the hardening, but the Bruner method may be used if desired. Unless protected by metal the surface edges of all slabs shall be rounded to a radius of one-half ($\frac{1}{2}$) in.

52. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course, and shall be mixed dry with the cement and aggregate until the mixture is of a uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

ONE-COURSE FLOOR.

53. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of portland cement to not more than two (2) cu. ft. of fine aggregate and not more than three (3) cu. ft. of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the coarse aggregate.

A cubic yard of concrete in place shall contain not less than six and eight-tenths (6.8) cu. ft. of cement.

54. *Consistency.*—The materials shall be mixed with sufficient water to produce a concrete which will hold its shape when struck off with a strikeboard. The consistency shall not be such as to cause a separation of the mortar from the coarse aggregate in handling.

55. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation competing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete brought to the established grade with a strikeboard. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. Workmen shall not be permitted to walk on the freshly-laid concrete. In no case shall concrete be deposited upon a frozen sub-grade or sub-base.

56. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having any dimensions greater than ten (10) ft., shall be reinforced with wire fabric or with plain or deformed bars. The reinforcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

57. *Finishing.*—After the concrete has been brought to the established grade by means of a strikeboard, and has hardened somewhat, but is still workable, it shall be floated with a wood float in a manner which will thoroughly compact it and provide an even surface. When required, the surface shall be steel troweled, but excessive working shall be avoided. Unless protected by metal the surface edges of all slabs shall be rounded one-half ($\frac{1}{2}$) in.

DISCUSSION.

Mr. Davis. MR. H. H. DAVIS (*by letter*).—With reference to the proposed Specification for Concrete Sidewalks and Floors, I desire to make the following comments and present some data to show that the part of these specifications relating to fine aggregates is inconsistent and unreliable as a means of selecting fine aggregates for use in concrete.

The phrase "consisting of quartzitic or other hard material" cannot be interpreted strictly, as it would exclude most stone screenings, and if loosely applied it adds nothing to the strength of the clause "sand or screenings from hard, tough, crushed rock or gravel."

Referring to that part of the specification which states for fine aggregates "that not more than 25 per cent shall pass a No. 50 sieve" and "not more than 5 per cent shall pass a 100-mesh sieve" the following data is presented to show that these limits as to gradation are inconsistent and unreliable indices of the strength of mortar or concrete in which the fine aggregates are employed. Most of this data is taken from Bureau of Standards Technologic Paper No. 58, and of some of the data considered I have personal knowledge as to details.

In Fig. 1 the variation in compressive strength compared with Ottawa standard sand of some 93 different natural sands is shown. Curves are drawn for the sands that conform to the specified limits as to size of particles and for sands that do not comply with the gradation requirements. All mortars are 1 : 3, by weight, and the strengths at 7 days, 4 weeks and 13 weeks are averaged for each sand. The data for these curves is found in Table 5, page 114, Technologic Paper 58.

Similarly in Fig. 2 the variation in tensile strength of some 139 natural sands as compared with similar tests of Ottawa standard sand in 1 : 3 mortars is shown. Curves are drawn to show the variation for both sands that do do not conform with the gradation requirements. The results for each sand of tests at 1, 4 and 13 weeks are averaged. The data for this figure is found in Table 7, page 121, Technologic Paper 58. The same cement was used in all tests noted above.

Likewise studying the strength tests of mortars containing stone screenings as aggregates (see Table 4, page 112, Technologic Paper 58) together with the granular analysis of these aggregates, the following facts are brought out. Of the 25 stone screenings tested, 22 do not comply with the gradation requirements of the suggested specification. The results of the compression tests of 1 : 3 mortars, however, show that all but three have a higher compressive strength than the average strength of 1 : 3 Ottawa sand mortar given in Table 5, above referred to. Of the three which have lower strength in mortar than Ottawa standard sand, one complies with the suggested gradation limits.

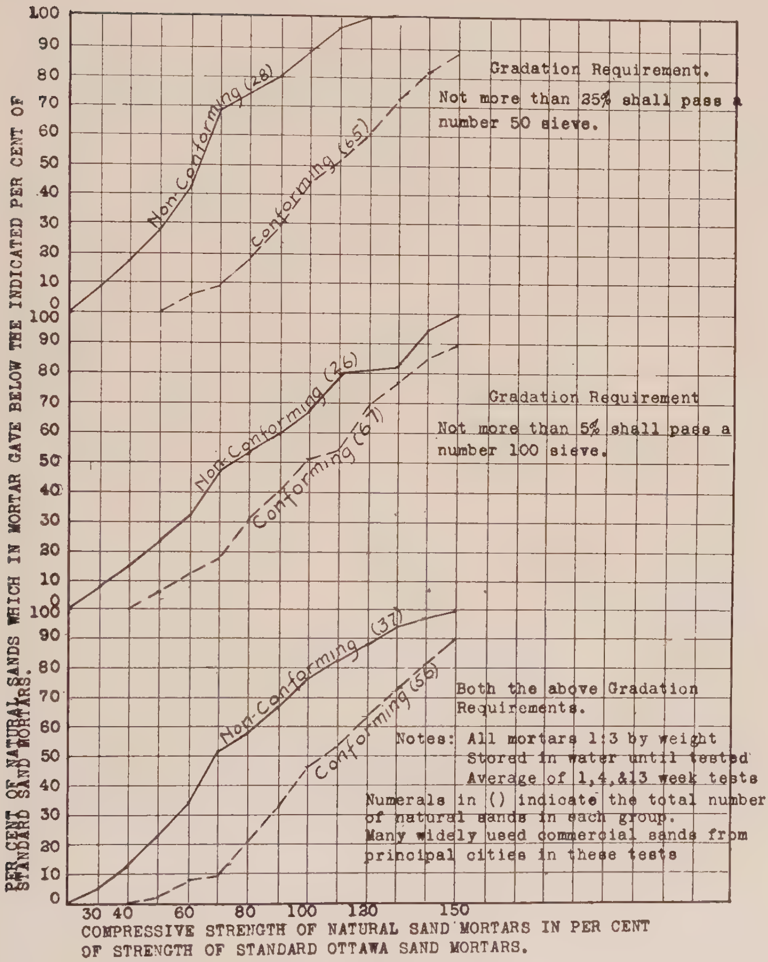

FIG. 1.—VARIATION IN COMPRESSIVE STRENGTH OF NATURAL SAND MORTARS AS COMPARED WITH STANDARD OTTAWA SAND MORTARS TO SHOW RELATION TO GRADATION REQUIREMENTS.

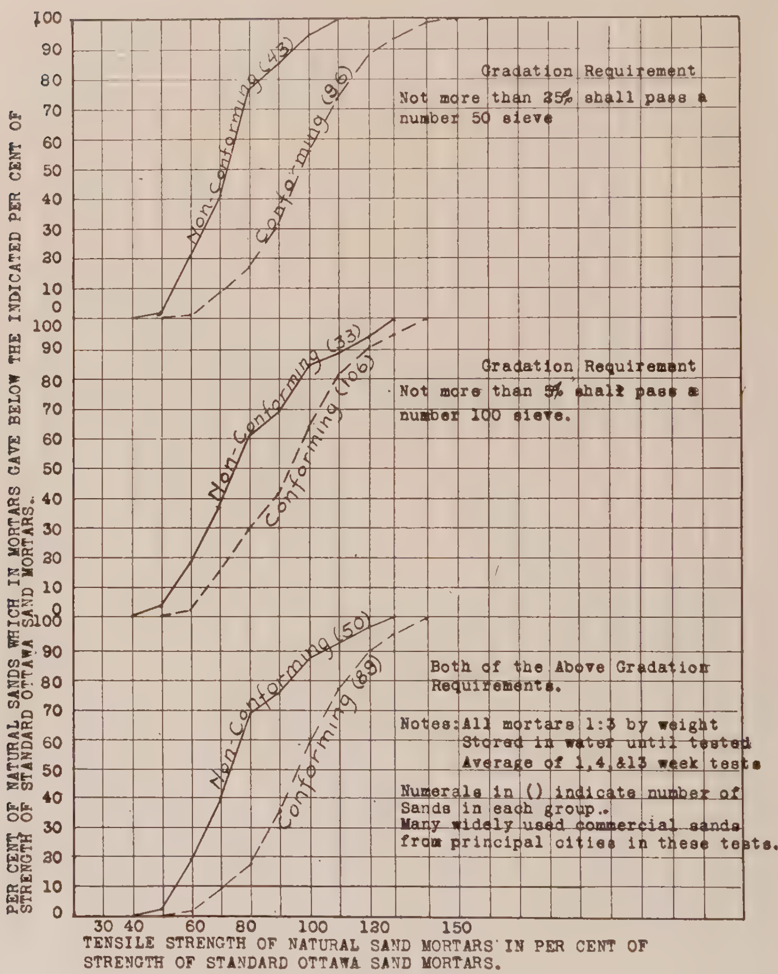

FIG. 2.—VARIATION IN TENSILE STRENGTH OF NATURAL SAND MORTARS AS COMPARED WITH STANDARD OTTAWA SAND MORTARS TO SHOW RELATION TO GRADATION REQUIREMENTS.

The following data give results of strength tests of concrete containing several coarse aggregates combined with different fine aggregates, some of which comply and some do not comply with the suggested gradation requirements. Mr. Davis.

Coarse Aggregate.	No.	Fine Aggregate. Character of Gradation.	Compressive Strength, lb. per sq. in. Tested at 4 Weeks.	
			1 : 2 : 4 Concrete.	1 : 3 : 6 Concrete.
No. 388* (limestone)....	184	Natural sand (yes)...........	2018	1471
	185	Natural sand (no)...........	1622	1053
No. 525† (trap rock)....	194	Natural sand (no)§...........	1840	965
	195	Natural sand (yes)‖..........	1765	1220
	429	Trap rock screenings (no).....	2298	1035
No. 526 (gravel)........	194	Natural sand (no)............	1600	896
	195	Natural sand (yes)...........	1825	985
	429	Trap rock screenings (no).....	1507	960
No. 508‡ (trap rock)....	200	Natural sand (yes)...........	2765	....
	201	Natural sand (no)............	2925	....

* From Table 23, page 154, Technologic Paper 58.
† From Table 26, page 156, Technologic Paper 58.
‡ From Table 31a, page 164, Technologic Paper 58.
§ Non-conforming.
‖ Conforming.

In this case, from superficial examination, sand No. 200 was considered the better and recommended, and used at a considerable increase in cost, on the work. The wisdom of recommendations based on appearance only seem questionable in view of the results of this and other similar strength tests when the coarse aggregate is well graded with considerable material between the ½ and ¼-in. size, the advantage of a coarse sand over fine sand is questionable.

From the above data it is felt that the statement that the suggested gradation requirements are in themself inconsistent and do not furnish a reliable index as to the suitability of the fine aggregate for use in mortar or concrete is justified.

It is my opinion that the present tendency in specifications for concrete or mortar aggregates is toward the direct test (preferably the compression test) of the mortar or concrete employing the aggregates that will probably be used in the work. All effort should be along the line of standardizing the manner of making such tests, and fixing a strength requirement for different proportions at different ages.

REPORT OF COMMITTEE ON CONCRETE SEWERS.

Your Committee on Concrete Sewers, in presenting "Proposed Standard Specifications for Monolithic Concrete Sewers and Recommended Rules for Concrete Sewer Design" wishes to put itself on record as considering this document purely as a progress report, and as a rough draft. For its own benefit the committee wishes to explain further that it has been unable to hold a single meeting during the past year on account of the wide separation of its membership. The committee feels that it has done a large amount of work under very serious handicap and finally decided, at the present stage of its work, to present the proposed specifications and rules in order to secure as much assistance as possible from the membership of the Institute.

The committee wishes to ask for as full discussion as is possible of the tentative draft presented, both at this meeting and by correspondence, and further asks that the committee be continued with the hope that by the succeeding meeting a second draft can be presented with a recommendation for adoption. Many of the clauses presented are in the nature of temporary compromises and undoubtedly will be considerably revised at any time the committee can hold a conference.

Respectfully submitted for the committee

W. W. Horner,
Chairman.

PROPOSED

STANDARD SPECIFICATIONS FOR MONOLITHIC CONCRETE SEWERS

AND

RECOMMENDED RULES FOR CONCRETE SEWER DESIGN.

PART I. MATERIALS.

Section 1. The materials required for the construction shall be furnished by the contractor and will be inspected by the engineer. Defective materials shall be removed from the site of the work and defective work repaired or replaced as directed. Facilities for the handling and inspection of materials and work at all times shall be furnished by the contractor.

If the work, or any part thereof, shall be found defective at any time before the final acceptance of the whole work, the contractor shall forthwith make good such defects in a manner satisfactory to the engineer.

CEMENT.

Section 2. All cement shall conform to the current Specifications for Portland Cement of the American Society for Testing Materials (Standard 1, American Concrete Institute).

Testing.

Section 3. Tests shall be made by the methods described in the current Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials (Standard 1, American Concrete Institute).

FINE AGGREGATE.

Section 4. Fine aggregate shall consist of sand, crushed stone or gravel screenings, graded from fine to coarse, and passing when dry a screen having holes one-fourth (¼) inch in diameter. It shall be clean, coarse, free from dust, soft particles, vegetable loam or other deleterious matter and not more than six (6) per cent shall pass a sieve having one hundred (100) meshes per lineal inch.

Section 5. Fine aggregate shall be of such quality that mortar composed of one part portland cement and three parts of fine aggregate by weight, when made into briquettes, will show a tensile strength at least equal to the strength of one to three (1 : 3) mortar of the same consistency made with the same cement and standard Ottawa sand. If the aggregate be of poorer quality, the proportion of cement in the mortar shall be increased to secure the desired strength. If the strength developed by the aggregate in the one to three (1 : 3) mortar is less than seventy (70) per cent of the strength of the Ottawa sand mortar, the material shall be rejected.

COARSE AGGREGATE.

Section 6. The coarse aggregate shall consist of crushed stone or gravel which is retained on a screen having one-fourth ($\frac{1}{4}$) inch diameter holes and graded from the smallest to the largest particles. It shall be clean, hard, durable and free from dust and all deleterious matter and soft, flat or elongated particles. For reinforced concrete arches or plain concrete arches less than six (6) inches in thickness, the maximum size of particles shall be such as will pass a screen having one (1) inch diameter holes. For inverts and plain concrete arches over six (6) inches in thickness, the maximum size of particles shall be such as will pass a screen having one and one-half ($1\frac{1}{2}$) inch diameter holes.

Section 7. Where crushed stone is used, it shall be made from rock having a crushing strength of not less than pounds per square inch and a specific gravity of not less than .

(The above paragraph is for use in locations where limestone or sandstone of a questionable value are common. It is recommended that the Engineer insert proper values after examining and testing local stones. If all available stone is suitable, the paragraph may be omitted.)

SAMPLES OF AGGREGATE.

Samples.

Section 8. Samples of fine and coarse aggregates which the contractor proposes to use shall be submitted to the engineer, if so required by him, for examination at least two weeks before the contractor commences to deliver the materials at the site of the work. Materials shall not be delivered until the samples shall have been approved, and as delivered they shall in all respects be equal to the approved samples. Samples of sand, of about one quart, shall be submitted in glass jars with stoppers, and samples of not less than one cubic foot of coarse aggregate in suitable boxes or other receptacles. All samples shall be plainly and neatly labeled with place from which taken, where proposed to be used, date and name of collector.

Measurement of Ingredients.

For the purpose of determining proportions of materials for concrete, each bag of cement containing 94 lb. shall be considered as containing one (1) cubic foot. Sand and coarse aggregate shall be measured loose in approved boxes.

WATER.

Water.

Section 9. Water shall be provided at the site of the work by the contractor, who shall pay for all connections to existing mains where available, and for all necessary piping along the work. Water shall be free from oil, acid, alkalis or organic matter.

HYDRATED LIME.

Hydrated Lime.

Section 10. If hydrated lime is to be used for waterproofing the concrete, it shall be brought on the work in original packages of about forty pounds each, with the name of the manufacturer plainly marked thereon. It shall be dry and free from lumps, unhydrated lime or material which would affect the strength of the concrete.

CONCRETE REINFORCEMENT BARS.

Section 11. Steel bars for reinforced-concrete sewers shall conform to the current specifications of the American Society for Testing Materials for (*A*) Billet-Steel or (*B*) Rail-Steel as follows and except as limited in subsequent sections.

(*A*) *Billet-Steel Bars.*

Section 12.

12.1 (*a*) These specifications cover three classes of billet-steel concrete reinforcement bars, namely: plain, deformed and cold-twisted. **Classes.**

(*b*) Plain and deformed bars are of three grades, namely: structural steel, intermediate and hard.

12.2* (*a*) The structural steel grade shall be used unless otherwise specified.

(*b*) If desired, cold-twisted bars may be purchased on the basis of tests of the hot-rolled bars before twisting, in which case such tests shall govern and shall conform to the requirements specified for plain bars of structural steel grade. **Basis of Purchase.**

Manufacture.

12.3 (*a*) The steel may be made by the Bessemer or the open-hearth process. **Process.**

(*b*) The bars shall be rolled from new billets. No re-rolled material will be accepted.

12.4 Cold-twisted bars shall be twisted cold with one complete twist in a length not over 12 times the thickness of the bar. **Cold-twisted Bars.**

Chemical Properties and Tests.

12.5 The steel shall conform to the following requirements as to chemical composition: **Chemical Composition.**

Phosphorus	Bessemer	Not over 0.10 per cent
	Open-hearth	" " 0.05 "

12.6 An analysis to determine the percentage of carbon, manganese, phosphorus and sulfur, shall be made by the manufacturer from a test ingot taken during the pouring of each melt, a copy of which shall be given to the purchaser or his representative. This analysis shall conform to the requirements specified in Section 12.5. **Ladle Analysis.**

12.7 Analysis may be made by the purchaser from finished bars representing each melt of open-hearth steel, and each melt, or lot of ten tons, of Bessemer steel, in which case an excess of 25 per cent above the requirements specified in Section 12.5 shall be allowed. **Check Analysis.**

* Change grade to intermediate or hard to suit individual design.

Physical Properties and Tests.

12.8 (*a*) The bars shall conform to the requirements as to tensile properties given in Table I.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

Modifications in Elongation. 12.9 (*a*) For plain and deformed bars over $\frac{3}{4}$ in. in thickness or diameter, a deduction of 1 from the percentages of elongation specified in Section 12.8 (*a*) shall be made for each increase of $\frac{1}{8}$ in. in thickness or diameter above $\frac{3}{4}$ in.

(*b*) For plain and deformed bars under $\frac{7}{16}$ in. in thickness or diameter, a deduction of 1 from the percentages of elongation specified in Section 12.8 (*a*) shall be made for each decrease of $\frac{1}{16}$ in. in thickness or diameter below $\frac{7}{16}$ in.

Bend Tests. 12.10 The test specimen shall bend cold around a pin without cracking on the outside of the bent portion, as given in Table II.

Test Specimens. 12.11 (*a*) Tension and bend test specimens for plain and deformed

TABLE I.—TENSILE PROPERTIES.

Properties Considered.	Plain Bars.			Deformed Bars.			Cold Twisted Bars.
	Structural-Steel Grade.	Inter-mediate Grade.	Hard Grade.	Structural-Steel Grade.	Inter-mediate Grade.	Hard Grade.	
Tensile strength, lb. per sq. in..........	55,000 to 70,000	70,000 to 85,000	80,000 min.	55,000 to 70,000	70,000 to 85,000	80,000 min.	Recorded only.
Yield point, min., lb. per sq. in..........	35,000	40,000	50,000	33,000	40,000	50,000	55,000
Elongation in 8 in., min., per cent......	$\frac{1,400,000}{\text{ten. str.}}$	$\frac{1,300,000}{\text{ten. str.}}$	$\frac{1,200,000}{\text{ten. str.}}$	$\frac{1,250,000}{\text{ten. str.}}$	$\frac{1,125,000}{\text{ten. str.}}$	$\frac{1,000,000}{\text{ten. str.}}$	5%

bars shall be taken from the finished bars, and shall be of the full thickness or diameter of bars as rolled; except that the specimens for deformed bars may be machined for a length of at least 9 in. if deemed necessary by the manufacturer to obtain uniform cross-section.

(*b*) Tension and bend test specimens for cold-twisted bars shall be taken from the finished bars, without further treatment; except as specified in Section 12.2 (*b*).

Number of Tests. 12.12 (*a*) One tension and one bend test shall be made from each melt of open-hearth steel, and from each melt, or lot of ten tons, of Bessemer steel; except that if material from one melt differs $\frac{3}{8}$ in. or more in thickness or diameter, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 12.8 (*a*) and any part of the fracture is outside

of the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

TABLE II.—BENDING TEST REQUIREMENTS.

Thickness or Diameter of Bar.	Plain Bars.			Deformed Bars.			Cold Twisted Bars.
	Structural-Steel Grade.	Inter-mediate Grade.	Hard Grade.	Structural-Steel Grade.	Inter-mediate Grade.	Hard Grade.	
Under ¾ in..........	180 deg. d—t	180 deg. d—2t	180 deg. d—3t	180 deg. d—t	180 deg. d—3t	180 deg. d—4t	180 deg. d—3t
¾ in. or over........	180 deg. d—t	90 deg. d—2t	90 deg. d—3t	180 deg. d—2t	90 deg. d—3t	90 deg. d—4t	180 deg. d—3t

EXPLANATORY NOTE: d—the diameter of pin about which the specimen is bent;
t—the thickness or diameter of the specimen.

(B) *Rail Steel Bars.*

Section 13.

13.1 These specifications cover three classes of rail-steel concrete reinforcement bars, namely, plain, deformed and hot-twisted. **Classes.**

Manufacture.

13.2* The bars shall be rolled from standard section tee rails. **Process.**

13.3 Hot-twisted bars shall have one complete twist in a length not over twelve times the thickness of the bar. **Hot-twisted Bars.**

Physical Properties and Tests.

13.4 (*a*) The bars shall conform to the following minimum requirements as to tensile properties:

Properties Considered.	Plain Bars.	Deformed and Hot-twisted Bars.
Tensile strength, lb. per sq. in....................................	80,000	80,000
Yield point, lb. per sq. in....................................	50,000	50,000
Elongation in 8 in., per cent....................................	$\frac{1,200,000}{\text{ten. str.}}$	$\frac{1,000,000}{\text{ten. str.}}$

See Section 13.5.

(*b*) The yield point shall be determined by the drop of the beam of the testing machine.

* For important work add to Section 13.2, "Bars shall be rolled only from the head of the rail."

(*The committee wishes to call attention to the necessity for particularly careful inspection and testing of rail steel in order that there may be no doubt of the uniformity of the product. Where billet steel can be obtained economically it should be used in preference to rail steel.*)

13.5 (*a*) For bars over three-fourths ($\frac{3}{4}$) inch in thickness or diameter, a deduction of one from the percentage of elongation specified in Section 13.4(*a*) shall be made for each increase of one-eighth ($\frac{1}{8}$) inch in thickness or diameter above three-fourths ($\frac{3}{4}$) inch.

(*b*) For bars under seven-sixteenths ($\frac{7}{16}$) inch in thickness or diameter, a deduction of one from the percentages of elongation specified in Section 13.4 (*a*) shall be made for each decrease of one-sixteenth ($\frac{1}{16}$) inch in thickness or diameter below seven-sixteenths ($\frac{7}{16}$) inch.

13.6 The test specimen shall bend cold around a pin without cracking on the outside of the bent portion, as follows:

Thickness or Diameter of Bar.	Plain Bars.	Deformed and Hot-twisted Bars.
Under ¾ in.	180 deg. d−3t	180 deg. d−4t
¾ in. or over	90 deg. d−3t	90 deg. d−4t

EXPLANATION NOTE: d=the diameter of pin about which the specimen is bent.
t=the thickness or diameter of the specimen.

Test Specimens.

13.7 (*a*) Tension and bending test specimens for plain and deformed bars shall be taken from the finished bars, and shall be of the full thickness or diameter of bars rolled; except that the specimens for deformed bars may be machined for a length of at least 9 in., if deemed necessary by the manufacturer to obtain uniform cross-section.

(*b*) Tension and bending test specimens for hot-twisted bars shall be taken from the finished bars, without further treatment.

Number of Tests.

13.8 (*a*) One tension and one bending test shall be made from each lot of ten tons or less of each size of bar rolled from rails varying not more than 10 lb., per yard in nominal weight.

(*b*) If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 13.4 (*a*) and any part of the fracture is outside of the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

GENERAL REQUIREMENTS.

Deformation.

Section 14. All bars are to be especially deformed uniformly throughout their entire length, either while being rolled or by subsequent twisting.*

Section 15. The Engineer may determine the efficiency of this deformation in providing mechanical bond by imbedding a specimen bar five inches in concrete made of one (1) part cement, two (2) parts sand and four (4)

* Where deformed bars are not obtainable, or on unimportant work, plain bars may be used, provided that proper allowance in bond is made in the design.

parts gravel. When the concrete is twenty-eight days old, the bar must sustain without slip such a load that, considering the bar as a round bar of equal weight per unit length, the bond developed per square inch of imbedded surface will be at least 225 lb.

Section 16. Dimensions of bars given on the plans are based on square sections. The net area and weights of bars used shall not be less than 95 per cent of the values for square bars as indicated. In computing the weights of steel, one cubic inch of steel shall be regarded as 0.283 lb.

Measurement and Payment.

Section 17. The quantity of metal to be paid for, shall be the number of pounds actually placed in accordance with the drawings or as ordered. It shall not include any waste metal due either to the nature of the construction or to the fact that the lengths supplied are too long or too short for their purpose.

The quantity paid for shall, however, include extra metal in laps, where authorized, due to the fact that a single bar would be unreasonably long.

All bars shall be of the length ordered and shall be in one piece where required up to 30 ft. in length.

Should the contractor be permitted to use shorter bars than directed, he shall provide the required lap and bear the expense of the extra steel and labor that are required.

The compensations shall cover the cost of furnishing and delivering metal, including any royalty, the cutting, bending, placing, fastening in position, coating with cement and all other work and materials connected therewith.

CASTINGS.

Description.

Section 18. The contractor shall furnish and place circular cast iron frames and covers for manholes and catch basins and any other iron castings shown on the drawings, or specified herein, necessary to complete the work.

Cast Iron.

Section 19. All castings shall be of tough, close-grained gray iron, free from blow-holes, shrinkage, and cold-shuts. They shall be sound, smooth, clean and free from blisters and all defects.

Workmanship.

Section 20. All casting shall be made accurately to dimensions to be furnished and shall be planed where marked or where otherwise necessary to secure perfectly flat and true surfaces. Allowance shall be made in the patterns so that the thickness shall not be reduced. Manhole covers shall be true and shall seat at all points.

Cleaning and Painting.

Section 21. All castings shall be thoroughly cleaned and painted before rusting begins and before leaving the shop, with two (2) coats of high grade asphaltum or other suitable varnish that the engineer may direct. After the castings have been placed in a satisfactory manner, all foreign adhering substances shall be removed and the castings given two (2) additional coats of asphaltum or other varnish as directed by the engineer.

Section 22. No castings shall be accepted, the weight of which shall be less than that due to its dimensions by more than five (5) per cent.

VITRIFIED BRICK FOR LINING INVERTS.

Section 23. All vitrified brick shall be uniform in size, and be not less than eight (8) inches by four (4) inches by two (2) inches, nor more than ten (10) by four and one-half ($4\frac{1}{2}$) inches by two and one-half ($2\frac{1}{2}$) inches in length, width or thickness, respectively. The brick shall be free from lime or other impurities, uniformly vitrified and annealed and shall have one edge face such that if the brick is laid on a horizontal plane on that face, no portion thereof shall be more than one-eighth ($\frac{1}{8}$) inch from the plane.

PART II. CONCRETE.

*Section 24.** Concrete shall consist of a mixture of cement, fine and coarse aggregates and water of the qualities hereinbefore specified.

Concrete shall be of three classes proportioned as follows:

Class.	Cement.	Fine Aggregate.	Coarse Aggregate.
A..	1 part	2 parts	4 parts
B..	1 part	2½ parts	5 parts
C..	1 part	3 parts	6 parts

The relative proportions of fine and coarse aggregates may be modified at the direction of the engineer, provided that the proportions of cement to the total of the aggregates measured separately shall not be changed.

Mixing. *Section 25.*† Concrete shall be machine mixed. The concrete mixer shall be designed to take one complete batch of materials (using whole bags of cement) and to mix that batch thoroughly before any portion of it is withdrawn from, or any portion of the succeeding batch is introduced. The mixer shall be equipped with a tank so designed that when once set it will automatically supply to the mixer the amount of water so determined. The mixer shall be equipped with an instrument for measuring the time of mix.

Section 26. Concrete shall be mixed at least *two minutes* after all the ingredients have been discharged into the mixer. The mixer shall not be run at a speed of more than ten (10) revolutions per minute and should the speed be less than ten (10) revolutions per minute, the time of mixing shall be increased in inverse proportion.

Section 27. No concrete shall be hand mixed except relatively small quantities and then only by special permission of the engineer.

Section 28. Where concrete is mixed by hand, the cement and fine aggregate shall be mixed dry on a properly constructed wooden or steel platform built for the purpose until it shall have obtained an even and uniform color throughout. This mixture shall then be spread to make a bed of uniform

* On important reinforced work or on work where a high degree of imperviousness is essential, add to Section 24 the following:

In addition to the constituents above, ten (10) pounds of hydrated lime shall be added for each sack of cement to be used.

† On small contracts or unimportant work, Sections 25 to 28 may be modified to permit of other types of mixers and to provide more generally for hand mixing.

thickness on which shall be spread the coarse aggregate and the whole wet with the required amount of water and turned with square pointed shovels at least three times or until a uniform mixture is secured, water being added from time to time if necessary. The contractor may use such other method or methods of mixing concrete by hand as the engineer may approve.

Rubble or Stone in Concrete.

Section 29. In all plain concrete, where the thickness is fifteen (15) inches or more, the contractor may embed broken pieces of sound stone, the greatest dimension of which does not exceed six (6) inches, and the least dimension of which is not less than three-quarters ($\frac{3}{4}$) of the greatest dimension. These stones shall be set in the concrete as the layers are being rammed, in a manner satisfactory to the engineer, and so placed that each stone is completely and perfectly embedded. In general, there shall be a space of four (4) inches between the stones and no stone shall come within four (4) inches of any exposed face. The stone shall be thoroughly cleaned and wet before placing.

Consistency.

Section 30. Concrete, except in special cases, shall be mixed wet enough to require no ramming and yet not so wet as to be "sloppy." Where comparatively dry mix is to be used, as in inverts, and near the crown of the arches, the concrete must be thoroughly tamped until the water flushes to the surface.

Work in Freezing Weather.

Section 31. Concrete shall not be mixed nor deposited in the work in freezing weather except when directed by the engineer. If the work on concrete structures is prosecuted in cold weather, proper precautions shall be taken for removing ice and frost from the materials, including heating the water and aggregates; for protecting the newly laid masonry from freezing, and for securing work satisfactory in all respects. Satisfactory covering for the newly laid concrete and such additional appliances and materials as may be required therefor, including steam pipes for keeping the air warm beneath the said covering shall be provided.

Transporting and Placing Concrete.

Transporting.

Section 32. Provision shall be made for quickly transporting the concrete from the mixer to the work and with as little shaking as possible, so that the tendency of water to rise to the top may be reduced to a minimum. Any concrete which may have been compacted during transportation shall be satisfactorily remixed before being placed in the work. Any concrete delayed one-half an hour in transit shall not be used in the work and must be removed from the premises.

Placing Concrete.

Section 33. Wet concrete shall be deposited so as to maintain a nearly level surface and avoid flowing along the forms. It shall be continuously and sufficiently worked to expel air and to force the aggregate away from the forms. In special cases, as where concrete is deposited on slopes, a comparatively dry mixture may be used but great care shall be exercised to spread such concrete evenly in layers not more than six (6) inches in thickenss and to ram it thoroughly. In general, the methods used shall be such as to give a compact, dense and impervious concrete with a smooth surface.

Joining New Work to Old.

Section 34. For the proper bonding of new and old concrete, such provisions shall be made of steps, dove-tails or other devices as may be required. Whenever new concrete is joined to old, the contact surface of the old concrete shall be thoroughly cleaned, using a stiff brush and a stream of water, if required, and shall be clean and wet at the moment the fresh concrete is placed. Where ordered, a thick wash of rich mortar shall be run over the contact surface of the old concrete. Where it is of importance that the joint between the new and old work shall be as strong and tight as possible, especial precautions shall be taken, such as picking off the top one or two inches of the old work so as to remove the laitance or washing the old cement off the surface with acid or alkali and later with water to remove all traces of them, or both, as may be required.

Finish of Concrete Surfaces.

Section 35. Special care shall be taken that all concrete surfaces shall be smooth and free from indentations or projections. All surfaces shall be free from voids, exposed stones and other imperfections. If such imperfections are found upon removing the forms, the faults shall be corrected at the contractor's expense by filling with mortar or otherwise, as directed, even to the extent of taking down and replacing unsatisfactory concrete.

Plastering of Concrete Surfaces.

Section 36. No plastering of any concrete surface shall be done unless expressly permitted and if so permitted shall be done in strict accordance with directions. No payment will be made for plastering done to correct defective work.

Masonry not to be Laid in Water.

Section 37. No concrete or other masonry shall be deposited under water without permission and then only in accordance with directions. The contractor shall not, without permission, allow water to rise on any masonry until the mortar shall have set at least twelve (12) hours.

Forms.

Forms.

Section 38. The contractor shall provide suitable collapsible centers or forms with smooth surfaces of ample strength and rigidly braced. The bracing shall be adequate to prevent deviations from the correct lines.

Section 39. The contractor shall submit the design of the forms to the engineer for approval when required, and shall have the forms erected complete in the shop where fabricated for the inspection of the engineer before shipment. All steel forms shall be neatly and accurately made with all smiliar parts in each longitudinal section of form interchangeable with other sections. Bent plates required to fit shall be rolled and fabricated to the correct curves before assembling. Suitable forms shall be provided for bends in the sewer. Steel filler plates shall be furnished.

Section 40. All wooden forms shall be built of clean, sound lumber, reasonably free from knots, dressed on all sides and neatly fitted. Tongued and grooved material shall be used where required. The form surface shall be watertight, securely fastened to the ribs or supports.

Section 41. No forms built up in the trench or ribs with separate pieces of wooden lagging, piece by piece, will be allowed except for specials or curves.

Section 42. No center or form shall be used which is not clean and of proper shape and strength and in every way suitable. Before placing concrete, the forms shall be coated with vaseline, form grease or other suitable substance, approved by the engineer, to prevent adherence of concrete.

Placing Reinforcement.

Section 43. The contractor shall furnish and place all steel bars required for concrete reinforcement of whatever size, shape and length required, including all cutting, bending and fastening and any special work necessary to hold them accurately in place and protect them from injury or corrosion. **Work to be Done.**

Section 44. All steel reinforcement shall be placed in the exact positions and with the spacing shown on the drawings or as ordered, and it shall be so fastened in position as to prevent displacement while the concrete is being deposited. **Placing Concrete.**

Section 45. The reinforcing steel shall be bent to the shapes shown on the drawings or as required. The ends of the bars shall be bent or hooked over if required. The length of the laps for bonding the adjacent bars shall not be less than thirty times the diameter of the bar, when the steel is designed for working stress of twelve thousand (12,000) pounds and not less than forty times the diameter of the bar when the steel is designed for working stress of sixteen thousand (16,000) pounds per square inch. Where the bars are of different sizes, the diameter of the larger bar shall be used. **Shaping and Splicing.**

Section 46. Steel must be stored in such manner that its condition will at all times correspond to that under which the samples were taken. **Storing.**

PART III.—GENERAL CONSTRUCTION.

*Section 47.** The width of trench for circular sewers shall be equal to the greatest outside width of the sewer. Below the springing line for such sewer, the trench shall be accurately shaped to the form of the outside of the masonry and the concrete shall have a firm bearing on the natural soil or rock at all points below the springing line. **Width of Trench.**

Section 48. At the direction of the engineer, the width of the trench for sewers of the horseshoe and similar types shall be one foot greater than the outside width of masonry to allow for satisfactory bracing.

Section 49. Underdrains of agricultural tile or vitrified pipe, laid in gravel or crushed stone, shall be constructed of the size, and where directed by the engineer for the purpose of keeping the work free from water during construction, such drains to be abandoned when the work is completed; underdrains so laid shall lead to sumps or manholes and water flowing to them shall be removed by pumping. Such pumping shall be carried on continuously, day and night, and the level of the ground water shall be maintained below any cement or concrete which may be placed in the work for a period of at least twelve hours after such cement or concrete is placed. When the **Underdrain.**

* Where there is a probability that wet ground will make the shaping of circular inverts impossible, an alternate section of a suitable type shall be used.

temporary underdrains above described are abandoned, they shall be cut and plugged where directed by the engineer and the sump holes above described shall be solidly filled with approved material.

Construction of Inverts.

Section 50. On all sections having a comparatively flat invert, the contractor shall first build the complete invert while on all circular sections, he shall build a center strip which shall not be less than one-fourth circumference. The invert or center strip shall be placed in sections of not over sixteen (16) feet where the surface is to be finished with end screeds and a longitudinal straight edge and not more than twenty feet if a vitrified brick lining is to be provided.

Granolithic Finish for Invert.

Section 51. Where required a granolithic finish shall be applied to the fresh concrete as soon as the condition of its surface will permit. This finish shall consist of a mixture of one part of cement to two parts of granite, trap rock or other hard rock chips, graded from one-eighth ($\frac{1}{8}$) inch to one-half ($\frac{1}{2}$) inch in size, with enough stone dust to permit of easily flushing to a smooth surface when laid one and one-half ($1\frac{1}{2}$) inches thick. The upper surface shall be formed by means of screeds and shall be floated and trowelled to a smooth surface. As soon as this surface is dry enough to receive it, a dry mixture of two parts of cement and one part of sand, free from crusher dust and particles larger than one-eighth ($\frac{1}{8}$) inch shall be sprinkled over it and then the surface shall be floated and trowelled. This treatment shall be repeated at least once, and where the proportion of very fine material in the aggregate necessitates it, a total of three dryer coats shall be applied. Where the placing of the dryer coat must be deferred until the day following the pouring of the concrete invert, the concrete shall be first moistened and covered with neat cement, which shall be thoroughly broomed into the concrete in the form of a thick paste.

Vitrified Brick Work.*

*Where required, the invert shall be lined with vitrified brick as shown on the plans. The concrete bottom shall be accurately shaped up to a line one-half ($\frac{1}{2}$) inch below the bottom of the vitrified brick and allowed to set before any vitrified brick are laid. Dry mortar made of one (1) part of Portland cement and three (3) parts of sand shall be spread on the finished surface to the depth required to bring the brick to the required line. The bricks shall be laid on edge in straight lines and in a workmanlike manner and so that all joints shall be broken by a lap of at least two (2) inches. After being laid, the bricks shall be rolled or tamped until every brick shall have a solid bearing and the top of the finished brick work shall present a smooth and even surface and conform accurately with the shape of the invert as shown on the plans. The joints between the brick shall be grouted with mortar made of one (1) part Portland cement and two (2) parts sand and the surface shall be brushed until every joint is completely filled. The bottom must be kept free from water until the brickwork is completed and no water will be allowed to run over the completed work until it shall have set.

Section 52. The unfinished surface of the invert on which the concrete of sidewalls or arches is to be placed, shall be made as rough as possible.

* This construction is particularly applicable to sewers having comparatively flat inverts, and is more difficult to carry out with circular sewers

In unreinforced work, dovetails shall be formed as provided in Section 34. In reinforced work, where projecting bars may interfere with the formation of dovetail joints, the invert concrete shall be thoroughly cleaned by a pressure stream of water or scrubbed and every precaution shall be used to prevent earth or material from the forms falling on the surface after cleaning.

Keeping Concrete Moist.

Section 53. Precaution shall be taken to prevent concrete from drying until there is no danger of cracking from lack of moisture. Concrete shall be kept moist for at least one week, unless sooner covered with earth. This may be done by a covering of wet sand, burlap, continuous sprinkling or by some other method approved by the engineer.

Section 54. Forms for slabs or very flat arches as in box sections or roofs of special chambers shall remain in place for at least seven days. No load shall be placed on the concrete for fourteen (14) days, and then only with the permission of the engineer.

Removal of Arch Forms.

Arch forms shall not be slackened until the backfilling has been carried to a height of at least one foot above the top of the arch and tamped. Arch forms shall remain in place for forty-eight (48) hours when conditions are most favorable for the hardening of the concrete and for a longer time, as the engineer may direct during inclement weather, or where unusual conditions exist. Permission for dropping center must be secured from the engineer for each arch unit.*

Section 55. Backfill, over and around arch sewers, shall be placed as soon as possible after the cement has set. The filling up to a plane two (2) feet above the top of the arch shall be made from the best earth and shall not contain a sufficient amount of large stones as to allow the pieces of stone to become wedged. It should be filled in layers of not over six (6) inches and carefully tamped. If the remainder of the backfill is dumped from buckets, the contents of the buckets should not be allowed to fall more than five (5) feet unless the impact is broken by timber grillage. Bracing should generally be removed only when the trench below it has become completely filled and every precaution shall be taken to prevent any large slips of earth from the side of the trench on to or against the green arch. All voids left by withdrawal of sheeting shall be immediately filled with sand, by ramming with tools especially adapted to that purpose, by watering or otherwise as may be directed.

Section 56. During the construction of the sewer, care should be taken that no loose mortar or concrete shall be allowed to remain on the interior surface of the invert. At the completion of the work all débris shall be removed and the invert shall be left clean and smooth.

GENERAL NOTE.

No attempt has been made to include herein detailed specifications, for much of the work entering into sewer construction, such as earth and rock excavation, sheeting and bracing, etc.

* For small arches, 6 ft. or less, and under the most favorable conditions, forms may be dropped in 24 hours.

No specification has been included in regard to the measurement of materials or the amount of work covered in any particular compensation, other than for reinforcing steel, as it has been considered best to leave these clauses to be worked out under the conditions prevailing on the particular piece of work.

RECOMMENDED RULES FOR CONCRETE SEWER DESIGN.

(To accompany the Specifications for Concrete Sewers.)

1. Concrete sewers without reinforcement are approved for sizes between 30 and 60 in. mean diameter. Plain concrete sewers between these sizes are to be used only in rock or hard soils. It is recommended that the minimum thickness for a diameter of 36 in. or under should be 5 in. and for a 5 ft. diameter 7 in. with intermediate sizes in proportion. These thicknesses are to be taken as a minimum for circular sewers and used only under favorable conditions.

2. All sewers near the surface and subject to moving loads or vibration, should be reinforced. For sewers of 6 ft. or less in diameter, it is recommended that the reinforcement be $\frac{1}{2}$ of 1 per cent in one ring placed at the inside at the crown, and at the outside at the springing lines.

If it appears at all possible that the horizontal pressures on the sewer might be large, use two rings of reinforcement.

3. It is recommended that for all sewers greater than 6 ft. in diameter, several possible types of loading be assumed and stresses be calculated on the elastic arch theory. (The methods are indicated in Tourneaure & Maurer's *Principles of Reinforced Concrete*, or in Metcalf & Eddy's *American Sewerage Practice, Volume I.*)

4. It is also suggested that in sewers of greater than 6 ft. diameter, it may be found economical to adopt a section having a comparatively flat bottom, and an arch with or without intermediate side walls.

5. The minimum thickness of concrete in sections of this type should be 8 in. This is recommended as a factor of safety against poor placing and also to secure the water-proof structure.

6. The specifications submitted provide for three classes of concrete. It is recommended that all arches be built of Class "A" concrete and that the inverts be of Class "A" concrete except in rock or very hard soils, where Class "B" concrete may be used.

7. For reinforced work in bad ground, the designer should provide for a raft of Class "C" concrete of from 4 to 6 in. in depth, which is to be allowed to set before the reinforced structure is started. This is advisable to facilitate good workmanship and particularly to prevent contamination of the concrete around the reinforcement by mud or sand.

8. The distance from the face of reinforcing steel to the face of the concrete should be not less than 2 in.

9. In determining dimensions of concrete and reinforcement, the following working stresses should be the maximum used:

(*a*) The maximum working stress in the steel where structural grade is used should be not more than 12,000 lb. and for intermediate or hard grades, or for cold twisted bars, 16,000 lb. per sq. in.

The maximum working stress in rail steel should not exceed 16,000 lb. per sq. in.

(*b*) The maximum working stresses in concrete are based on the Report of the Joint Committee on Concrete and Reinforced Concrete and are about 25 per cent less than the stresses there recommended.

Working Stresses in Pounds per Square Inch.

Aggregate.	Class A.	Class B.	Class C.
Granite or trap rock	550	450	350
Gravel or hard limestone	500	400	325
Soft limestone or sandstone	375	300	250

Class "B" concrete is not recommended for use in the sewer proper. Soft limestone and sandstones are prohibited if the accompanying specification is rigidly carried out.

These stresses should be further reduced where construction conditions are likely to be very unfavorable to good workmanship, as in very wet or very deep trenches.

10. In all important work, specify that the reinforcement shall be held in place with steel chairs or holders and wire ties.

11. Attention is called to the fact that with sewers having comparatively flat inverts, careful consideration must be given to the distribution of load across the invert. Where soils are likely to be compressible, the weight should be taken as uniformly distributed. The stresses in such inverts should be carefully analyzed, as they are generally more severe than in the other parts of the sewer.

It will generally be advisable to provide alternate details of the invert for use in rock cuts, when resting on rock or nearly incompressible soils and for soft or wet ground.

12. Accompanying specifications provide for either a vitrified brick invert lining or a granolithic finish on the invert. The committee recommends the former if the velocities in the sewer are over 10 ft. per second, or if the flow will carry a large amount of sand or grit, or in the case of wastes, which might be detrimental to the concrete. The committee also calls attention to the fact that a smooth granolithic finish is difficult to secure under wet trench conditions.

REPORT OF COMMITTEE ON BUILDING BLOCKS AND CEMENT PRODUCTS.

Your committee early recognized that the entire field of concrete products could not be fully covered in the time at the disposal of its several members. While several branches of the concrete industry have been investigated and information obtained, at the same time it was the opinion of the committee that greater results might be obtained by confining their efforts to a thorough investigation of a few of the different branches of concrete products manufactured. Believing that the membership of the American Concrete Institute, and especially those members who are interested in the manufacture of concrete products, would be able to form a clearer conception of the magnitude and possibilities of this industry, a few historical facts regarding the development of concrete blocks and building units are presented herewith. This committee collected and tabulated a large amount of manufacturing data which appears in this report. Specifications for certain products are recommended for adoption.

Concrete Building Units.

Those who have carefully followed the activities of the American Concrete Institute, from the date of its inception, have, no doubt, been surprised by the wonderful strides made in the development of the concrete products industry.

The production of concrete building blocks is by no means a new development, but their extensive manufacture was not undertaken until within the last fifteen years. Reference to some of the old files of engineering journals show that concrete blocks were made soon after the discovery of portland cement. These early blocks, however, were of large section, and made in very crude wooden forms. With the development of the industry it was a natural sequence that mechanical equipment should be provided for the manufacture of a commodity of this kind, and it has only been during recent years that machines were developed and have reached their present high standard of efficiency.

There is an ever-increasing demand for the use of concrete in ornamental and trim work, together with high-grade building units in textures that are truly original with concrete and of a distinctive type. The concrete block manufacturer is no longer striving to imitate other building materials, but is endeavoring to place concrete in a class of its own by treating it as a distinctive structural material of great merit.

The present day commercial manufacture of concrete blocks, building units, and dimension stone, has developed into three processes. These are generally known as the tamped, pressed and cast methods.

In making tamped units a concrete of comparatively dry mixture is used, placed in the mold and by tamping either by hand or automatic equipment

fills the mold with a dense concrete. The amount of water used in this method admits of a great variation, depending more or less upon the equipment in use and the nature and type of product being made.

Where concrete units are made by the pressed, or pressure process, a wetter mixture is employed, is placed in the receptacles or molds and by means of external pressures, in many instances running up to thousands of pounds, is formed to the desired shape by means of hand or power equipment.

The cast, or poured process, as the name implies, makes use of concrete of a consistency that permits of easy flow and handling. This concrete is poured into molds, of iron, gelatine, plaster or sand, and is puddled in order to thoroughly fill all parts of the molds with a dense concrete.

There are a large number and variety of machines on the market today for the making of tamped concrete units. The machines range in size from a small hand machine to large power driven and entirely automatic equipment with correspondingly large output.

There likewise exists a variety of equipment for the manufacture of concrete units by means of a pressure method.

The equipment necessary for the utilization of the third, or cast process, consists of the necessary metal molds or molds of plaster, gelatin or sand, which, in many instances, are being made by the manufacturer, himself.

Many of the modern, well-equipped concrete products plants use, at various times, all of these three processes. For example, a large portion of trim and dimension stone will be manufactured by tamping, using wooden or plaster molds for the purpose. Plain building units are made by either the tamped or pressed process, but in the manufacture of ornamental and decorative units, it is almost imperative that the third, or cast method be employed; especially is this true where the units are undercut or highly ornamented. Each of these several processes has many advantages which should not be overlooked. The enterprising manufacturer of concrete products should consider these methods without prejudice, and after careful study and investigation, select that type or types best suited to his needs.

The early attempts of the concrete block manufacturer to imitate some other natural building material resulted in an entirely different manner than that which had been anticipated. Architects became prejudiced against the early forms of concrete blocks because of their crude imitation of pitched face stone. To overcome such objection, however, the manufacturers of concrete block machinery developed equipment and methods for the production of units with other surfaces, such as bush-hammered, paneled and tooled. These effects are easy to produce and are far more attractive.

The greatest forward step in this respect has been the production of surfaces made of specially selected aggregates from which the surface film of cement has been removed by special treatments.

This has opened a new and unlimited field to the concrete products manufacturer. Architects find that ornamental and decorative concrete of this texture lends itself particularly to their requirements. The surface of film cement is removed by spraying the surface of the unit as soon as it has been made with a fine water jet, or scrubbing it after partially, hardening with

a mixture of water and acid. The judicious selection of aggregates and th e proportioning with either white or gray portland cement offers a wide range of effects in color and texture.

The concrete products manufacturer should study this development and work with the architect to further the use of concrete units in structural and architectural work.

CONCRETE PIPE AND DRAIN TILE.

Concrete pipe has been manufactured for a number of years, although the wonderful development in its manufacture and use can really be traced to the last ten or twelve years. Three large fields have been developed for the use of concrete pipe, namely, sewers, drainage and culverts, and irrigation work.

As applied to sewer installations, concrete pipe is used in sizes from 4 to 96 in., the smallest sizes up to and including 24 in. being usually of machine manufacture. Pipe larger than 24 in. in diameter is generally made by hand and sometimes at the site of the work. It has been used in both sanitary and storm sewers in over 200 representative cities of this country and Canada, and, with the possible exception of a few isolated cases, has been uniformly satisfactory. Where difficulties have arisen in the use of concrete pipe, the fault has been traced not to a just criticism of the concrete pipe as a material, but rather to the manufacturing methods used in connection with the particular pipe in question. From $3,000,000 to $5,000,000 worth of sewer work is done in this country every year. This indicates the possibilities for the use of concrete in sewer construction.

In the use of concrete pipe for land drainage and culverts, most rapid and wonderful strides have been made during the past twelve or fifteen years. Antiquated methods have been superseded by modern equipment operating on efficient lines. As a result of improved quality of the product, concrete is taking a leading place in drainage work. As to the possibilities offered for the use of concrete drain tile, we cite the instance of one state where, at the present time, over $5,000,000 worth of drainage work is under contract, and it is anticipated that within the next four or five years, $20,000,000 more drainage work will be undertaken. A conservative estimate places the cost of tile used in drainage work at 40 per cent for the total expenditure. The possibilities of land drainage are indeed very great in not one, but many states of the Union, and the field presented for the manufacture of concrete drain tile is almost unlimited.

Concrete drain tile is made in sizes from 4 to 60 in. in diameter, the smallest sizes being manufactured on machines capable of producing a large output, while the extremely large sizes are hand-made. Many tests have been conducted by various institutions, which tests have shown conclusively the manifold advantages possessed by tile of this material.

In the western part of this country an extensive field has been opened for the use of concrete in connection with irrrigation. Concrete flow lines are replacing the open ditch with its attendant high water losses and high expense

for maintenance. The government has made extensive use of concrete pipe, in some instances employing it to withstand considerable internal pressure.

STATISTICS OF CONCRETTE PRODUCTS INDUSTRY.

An investigation conducted among the manufacturers of concrete products discloses some figures which are truly remarkable, and at the same time gives those unfamiliar with the concrete products industry a fair conception of the extent to which concrete products are manufactured in this country. It must be borne in mind, however, that while these statistics must of necessity be somewhat incomplete, due to difficulties encountered when obtaining the information from many of the concrete products manufacturers, they nevertheless furnish a basis for studying the magnitude of this industry. The results of the investigation are summarized in the accompanying table.

Products.	Plants Reporting.	Average Size.	Average Selling Price.	Annual Capacity.
Blocks (building)	253	8 x 8 x 16 in	10 to 25 cts	22,926,600 cu. ft.
Blocks (silo)	98		40 to 60 cts	2,173,000 cu. ft.
Brick	108	Standard	$10 to $30 per M	55,848,000
Drain tile	111	4 to 24 in	$15 per M. up	48,034,103 ft.
Ornamental	136		25 cts. up	17,712 cu. ft.
Posts	115	No standard	25 cts. up	381,755
Roofing tile	22		$5 to $7.50 per sq	13,200 squares
Sewer pipe	60	5 in. up	$25 per M. up	3,635,750 ft.
Staves (silo)	32		30 cts. up	708,500
Structural tile	8		$28 per M	22,500,000
Vaults	11		12 up	
Well curbing	3	30 x 36 in		
Shingles	2			
Culvert pipe	9	15 in. up	$200 per M. up	174,000 ft.
Catch basins	3			
Railroad ties	1			

SPECIFICATIONS.

Your committee early recognized the fact that lack of proper specifications was a severe handicap to the development of the proper demand for concrete products. Attached to this report and made a part thereof are specifications and building regulations for the manufacture and use of concrete architectural stone, building blocks and brick, also specifications for concrete fence posts. The committee recommends their adoption and submission to letter ballot as Standards. This committee also recommends the adoption of the Standard Specifications for Drain Tile as adopted by the American Society for Testing Materials at its annual meeting, June, 1916.*

It is the sense of this committee relative to the specifications for sewer pipe that the American Concrete Institute await the final action of the Committee on Clay and Cement Sewer Pipe of the American Society for Testing Materials who are making an investigation of this subject.

The committee also submits a few views of concrete products, which are reproduced herewith.

* These specifications are all printed in this volume as Standards, having been so adopted by letter ballot canvassed April 10, 1917.—EDITOR

FIG. 1.—RESIDENCE OF GRANITE FACED CONCRETE BLOCKS AND CONCRETE COLUMN, CAPITALS AND SPINDLES.

FIG. 2.—DARK BLOCKS ARE FACED WITH GRAY GRANITE AND GRAY CEMENT, WHITE STONE OF WHITE CEMENT, SAND AND MARBLE DUST.

FIG. 3.—TRIM STONE FACED WITH WHITE CEMENT AND GRAY GRANITE.

FIG. 4.—SAMPLES OF CONCRETE PORCH ORNAMENTS.

Respectfully submitted,

ROBT. F. HAVLIK, *Chairman*,
C. M. WOOD, *Secretary*,
D. A. ABRAMS,
V. D. ALLEN,
P. H. ATWOOD,
J. K. HARRIDGE,
P. E. MCALLISTER.

DISCUSSION.

Mr. Chapman.

Mr. Cloyd M. Chapman.—In the fence post specification I object to the requirement that every test shall be a destructive test. The requirement that the post shall, under certain conditions, sustain a load of 1000 lb. I think should be a sufficient requirement to pass the post without going further to a destructive test. I do not see anything to be accomplished by breaking every post you test. The specification here for fine aggregate is one that I do not think can be enforced. I do not know of any other rock of sand form in nature that is as hard as quartzite or quartzitic grains except quartzite. The specification that not more than 30 per cent shall pass or shall be finer than a 50-mesh sieve is all right as applying to sands but not as applying to screenings.

Mr. Abrams.

Mr. D. A. Abrams.—As a member of the committee and somewhat responsible for this specification, I think it may be well to make a few remarks on Mr. Chapman's statement. Practically all tests are destructive. If we want to test a car bolster or what not, we test it to destruction; that is the only way we find out what it is good for, and the small saving of 25 or 30 cents per post is scarcely worth making, in view of the information we can get from a destructive test. Mr. Chapman is a little too strict in his interpretation of what we call quartzitic material. It was not the intention that quartz sand or quartzite only be admitted under these laws, but that term is used in a descriptive manner that is quite common in specifications of other kinds, merely as indicating the nature of the material aimed at rather than attempting to give an exact description of it. Although this specification for fine aggregate is quite common, is much similar to what we have in other specifications, I should be the first one to grant that it is not satisfactory and I think that I can flatter myself by believing that I can write a better one, but I do not think we are ready to do it now, consequently we have to leave that for a while longer.

Mr. Woolson.

Mr. Ira H. Woolson (*by letter*).—I am voting no upon the proposed specifications referring to hollow concrete building blocks. My objections to these specifications are limited to Section 9, "Limit of Load." I consider that permission to load a hollow building block to 167 lb. per sq. in. of gross area, which is only requiring a factor of safety of 6 on its required test specification of 1000 lb. per sq. in., is very unwise.

A factor of safety of 10 upon brick, stone, tile, hollow blocks, etc., has been recognized for years in building work as proper, and is the requirement in the majority of building codes. Why it should be reduced to 6 for this particular material I cannot understand.

The material has a well-earned reputation for variability and quality owing to the sporadic character of its manufacture by irresponsible and unqualified persons. This fact of itself should be sufficient reason for main-

taining a factor of safety in its use as high as that which is commonly accepted for brick which is much more uniform in its quality. **Mr. Woolson.**

I also question the advisability of allowing a load of 300 lb. per sq. in. upon filled blocks. If the committee which drafted these specifications can produce reliable test data showing that well-seasoned blocks, afterwards filled with concrete, will increase its carrying capacity 75 to 80 per cent as indicated by this increased loading, I would be willing to accept the proposed allowable stress, and vote in favor of it; but until I have such information, I should certainly oppose it.

Records of such tests upon filled blocks such as I have been able to secure do not indicate that increased strength is obtained by filling; at least not sufficient to warrant any allowable increase in the working stress. Such filling undoubtedly adds to the stability of the wall, but I should like to have some reliable proof that it adds to the strength to any such extent as indicated in this proposed specification before I would vote for it. I have personally conducted a large number of tests upon this product, and I know its variable character as usually produced in back-lot manufacture.

Such blocks have another serious defect; namely, that in case of a fire in a building having such walls, the walls almost invariably crack through the webs parallel to the face of the wall, with the result that the walls are so weakened they either fall down, or must be taken down. You will recognize at once that there would be no salvage in such a wall, and for that reason insurance organizations usually allow very little, if any, credit in their insurance rate on a building in which they are used.

I think for the benefit of the industry it would have been wise to have introduced some specification to eliminate, or at least restrict, this defect if that were possible.

I am not unduly prejudiced against hollow concrete blocks, for I know that such blocks have been made and can be made which will render very satisfactory service. But there is no denying the fact that the material as a class is in bad odor with architects, builders, and the public in general, and for this very reason in standardizing the material everything possible should be done to restore it to favor.

REPORT OF COMMITTEE ON CONCRETE ROADS AND PAVEMENTS.

Last year your committee recommended certain changes in the Specifications for Concrete Roads, Streets and Alleys as shown on pages 433 to 446, inclusive, of the Proceedings of the American Concrete Institute for 1911. These changes were not presented in time to be printed and distributed 30 days before the 1916 annual meeting and therefore could not be submitted to letter ballot of the Institute. These changes come up formally at the 1917 Convention to be submitted to letter ballot.*

Your committee recommends additional changes as follows:

In Standards 5, 17, 18 and 19 change Clause 1 to read as follows:

"1. *Cement.*—The cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement, adopted by the American Society for Testing Materials, September 1, 1916, with all subsequent amendments and additions thereto adopted by said Society and by this Institute. (Standard No. 1.)

In Standards 5, 17, 18 and 19 change last sentence in second paragraph, Clause 2, to read: "If there is more than seven (7) per cent of clay or loam, by volume, in one hour's settlement after shaking in an excess of water, the material represented by the sample shall be rejected;" change last sentence in third paragraph, Clause 2, to read: "In other respects all briquettes shall be made in accordance with the methods outlined in the Standard Specifications and Tests for Portland Cement adopted by the American Society for Testing Materials, September 1, 1916;" change maximum size of coarse aggregate in last paragraph, Clause 2, from "1½ inches" to "2 inches."

In Standard 5, Clause 8, change to read as follows:

"*Shoulders.*—(Materials for the construction of shoulders shall be here described as desired by the engineer.)"

In Standards 17, 18 and 19, Clause 14, change the word "readjusted" to "adjusted."

In Standard 5, Clause 15, change last paragraph to read as follows:

"At the time of acceptance of the road or when concreting is discontinued for the winter, the ditches must be in finished condition with clean slopes and bottom containing no obstructions to the flow of water."

In Standard 5, Clause 16, and Standards 17, 18 and 19, Clause 15, change first paragraph to read as follows:

"*Construction.*—The subgrade shall be brought to a firm density by rolling the entire area with a self-propelled roller. All portions of the subgrade which are inaccessible," etc.

Omit second paragraph regarding sprinkling the subgrade with water.

In Standards 17 and 18, change second sentence in Clause 20 to read as follows:

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* These specifications as adopted as Standard, April 10, 1917, are printed elsewhere in this volume.—Editor.

"The finished surface shall conform to the lines as shown on the plans attached hereto."

In Standard 5, Clause 22, and Standards 17, 18 and 19, Clause 21, add the following:

"All catch basins, manhole tops, poles or other fixed objects which project through the pavement shall be separated from the concrete by joint filler."

In Standard 5, Clause 27, and Standards 17, 18 and 19, Clause 26, omit next to the last sentence which reads, "The drum shall revolve at a rate of speed," etc.

In Standard 5, Clause 30, Standards 17 and 19, Clause 29, and Standard 18, Clause 28, before the word "sufficient" in the first sentence, add the word "only."

In Standard 5, Clause 31, and Standards 17, 18 and 19, Clause 30, add the following to the note:

". . . and in most cases the use of reinforcement in pavements 16 feet wide or over is good practice."

In Standard 5, Clause 33, and Standards 17 and 19, Clause 32, change first three sentences in first paragraph to read as follows:

"*Finishing.*—The concrete shall be brought to a proper contour by any means which will insure a compact dense surface. Any holes left by removing any material or device used in constructing joints shall be filled immediately with concrete from the latest batch deposited."

Change second paragraph by omitting first sentence, and change second sentence to read as follows:

"Concrete shall be finished in a manner thoroughly to compact it and produce a surface free from depressions or inequalities of any kind."

In Standard 18, Clause 36, change first three sentences in first paragraph to read as follows:

"*Finishing.*—The wearing course shall be brought to a proper contour by any means which will insure a compact dense surface. Any holes left by removing any material or device used in constructing joints shall be filled immediately with concrete from the latest batch deposited."

Change second paragraph by omitting first sentence, and change second sentence to read as follows:

"The wearing course shall be finished in a manner thoroughly to compact it and produce a surface free from depressions or inequalities of any kind."

In Standard 5, Clause 33, omit last paragraph reading as follows:

"The edges of the pavement shall be rounded," etc.

In Standard 5, Clause 33, Standards 17 and 19, Clause 32, and Standard 18, Clause 36, add the following note:

NOTE.—It is recommended that the contractor be required at the end of each day's work to stamp in the surface of the concrete with letters $1\frac{1}{2}$ to 2 inches high and $\frac{1}{4}$ inch deep, the date and his name."

From drawings accompanying Standards 17 and 18 remove words "arc of circle," also change blind drain so that it does not come in contact with the concrete.

A. N. JOHNSON,
Chairman.

DISCUSSION.

Mr. Ashton. MR. ERNEST ASHTON.—I understand that the first change suggested by the committee was that relative to a field test with regard to measuring the amount of silt. It seems that the wording presented that calls for an excess of water is not sufficiently specific; to some engineers, an excess of water might mean two or three centimeters more than required to fill the void content, while another man might put in 50 per cent more water than actually required to fill the voids. To use 90 c.c. of water with 100 c.c. of sand, he would have five-ninths of the surface in the measure above the surface of the sand, whereas if another engineer would come along and use 300 c.c. of water, he would get 75 per cent of silt. It seems to me that 100 per cent of water in excess should be satisfactory.

Prof. Hatt. PROF. W. K. HATT.—We have before us Standard 17 for Concrete Roads, Streets and Alleys; Standard 18 for Two-Course Concrete Street Pavement; Standard 19, which is for a One-Course Concrete Alley Pavement, and Standard 20 for Paving Between Street Car Tracks; and all of these are up for discussion and disposition. The committee has reconsidered these specifications and now presents some new amendments. Those new amendments are, in brief, providing for rejection of fine aggregates when the amount of clay or loam exceeds 7 per cent; second, that the size of the coarse aggregate is increased from $1\frac{1}{2}$ in. to 2 in., third, that the thickness of the concrete at the sides of the road shall not be less than 6 in., when it formerly was 5; fourth, that the clause specifying the number of revolutions per minute of the drum is omitted in the concrete mixer; and fifth, that a note is added to the reinforcing, saying that it is good practice to reinforce roads over 16 ft. in width.

As I understand it, Mr. Chairman, those are the only substantial changes recommended this year in the specifications of the year before. The reinforcing of concrete roads was changed last year from that adopted by the Society. There was a specification requiring so many square inches longitudinal and so many square inches transverse. That was changed last year to a certain amount of weight per square foot. I think anybody who has been in contact with the controversy concerning the method of computing the number of square inches in concrete reinforcement, transverse and longitudinal, will agree that this is a proper move, to state the weight per square foot. In last year's specifications, there were very detailed instructions given for finishing this surface of the pavement. Since that time we have had other methods adopted, probably giving better results than the old hand finishing, and therefore it seemed to the committee best to remove those definite directions and allow a man to obtain the results by whatever method he sees fit.

Those, as far as I know, Mr. Chairman, are the main changes in our

standards. Discussing these in detail, I do not like some of the words in this specification for fine aggregates. It says that fine aggregate shall not contain vegetable or other deleterious matter or more than 3 per cent by weight of clay or loam. I think it is a mistake to try to write a dictionary on the floor of a convention; at least those who tried it have given up the task, but it seems to me that loam is a word that necessarily describes something with vegetable matter in it. I would prefer some other word than loam, or perhaps strike it out entirely. Then I would admit 3 per cent of clay and would not permit any vegetable matter of any kind. **Prof. Hatt.**

MR. RICHARD L. HUMPHREY.—I would like to discuss the chapters on reinforcing. I think the question of the reinforcement of concrete pavements and roads is one of the most important subjects that this convention has to consider, and I certainly hope that the paragraph as recommended by the committee will not be adopted by this Institute, for the reason that it is, in my opinion, wholly inadequate and not what this Institute should adopt at this time, in view of what we know about reinforcement. It seems to me that the specification changing from an area per width and length of the pavement to the weight per square foot was distinctly a step backward. The form in which some reinforcement is fabricated is such as to give a very unequal distribution of the metal that can not be controlled by the weight per square foot. I cannot conceive of anyone designing a floor slab or any slab by figuring the amount of weight of reinforcement it should have, or reinforcing a slab on the basis of the weight of metal in a square foot. The function of reinforcement is to prevent the spreading of cracks caused by settlement and changes due to contraction and temperature. The reinforced concrete slab is a mat on which traffic moves and, therefore, should be reinforced with an idea of giving it the character of a plate, and as a plate it should have a definite amount of reinforcement in both directions. While it is true, that slab does not have to meet the conditions of a floor, for the reason that it is supported more or less all over its surface, nevertheless, it is essentially a plate and the amount of metal used should be a function of the stresses in the direction of length and width and cannot be determined by an arbitrarily assumed weight per square foot, having no control over the amount and area of the reinforcement per foot of length and width of the pavement. **Mr. Humphrey.**

PROF. W. K. HATT.—Mr. Humphrey has advanced the analogue of the design of a floor slab, and he says that we figure out the number of square inches and the direction of the load to make that slab safe, and therefore we ought to do the same thing in a road slab. I disagree with him entirely. If you designed road slabs on that basis, I think the cost would be prohibitive. As a matter of fact, we get, in our road slabs, less than 0.1 per cent reinforcement; that is not a structural reinforcement. If you want any sort of structural reinforcement to carry loads, it ought at least to be 0.5 per cent. I do not think that the steel is put in concrete roads primarily as a structural reinforcement to relieve the concrete from tensile stresses, and as a matter of cold logic. I cannot see why we should have more reinforcing for a 20 ft. road than for a 16 ft. road. **Prof. Hatt.**

What that reinforcing does, as I see it, is this: I think we must admit

Prof. Hatt. that under conditions of unequal foundation, of caving, of frost, of cantilever loads, when the concrete road sticks out from either side of an old hard gravel or stone road, the concrete road being wider, there will be cracking. I am not speaking about temperature changes now, I am speaking about the cracks from the loads. What the steel does is to hold those two cracked sections together and hold this aggregation of slabs and asphalt and steel reinforcement together. After the crack fills up with dirt, it does not become very objectionable. That is one function of reinforcing. For that purpose, let us specify so many pounds per square foot and we will get as near to it as any other way.

With respect to the function of reinforcing in taking care of shrinkage and temperature strains, I do not know what the theory is, I do not know how much, what the percentage should be and the depth of slab to which you should compute that percentage. If we are going to wait until we know about that, I think we will not pass any specifications for some time.

REPORT OF THE COMMITTEE ON REINFORCED CONCRETE AND BUILDING LAWS.

At the meeting of the American Concrete Institute in Chicago a year ago your committee submitted a Proposed Standard Building Regulation for the Use of Reinforced Concrete. The action of the convention was to refer the report back to the committee for further consideration, for resubmitting at this convention.

The Standard Building Regulation has been reconsidered, revised, printed and distributed to the members of the Institute. Your committee has received and considered several written discussions of the printed report and at the committee meetings held at Chicago Feb. 7 and 8, 1917, decided to recommend certain changes and additions to the proposed regulation as preprinted. The revised regulation is printed following this report.

Mr. Edward Godfrey, member of the committee, was not present at the meetings of the committee on Feb. 7 and 8, 1917, but with regard to the Standard Building Regulations as printed, he has asked the chairman to report the following dissenting note.

> Mr. Godfrey dissents from the report in the matter of unit stresses recommended wherever the same are higher than those recommended by the Joint Committee. This in the interest of uniform standards and of safety in design.
>
> Mr. Godfrey also dissents from the report in the matter of bending moments to be figured for flat slabs wherever the same are less than those given in the Joint Committee report. The latter report already allows liberal indulgences to flat-slab builders by reducing the theoretical and actual bending moments. Even this reduction can only be justified by virtually depending upon tension in the concrete and by a knowledge of the fact that failure through that tension is not so probable in the case of flat construction as in other slabs and in girders.
>
> Mr. Godfrey also dissents from the report in the matter of provision against diagonal tension or shear failures. He would give stirrups and short shear members no recognition, for the reason that he holds that they have not shown themselves to have any definite value in tests and that analysis fails to show that any definite value can be ascribed to them; he also believes that dependence on stirrups to take end shear has resulted in much unsafe construction, and some failures. He would take care of diagonal tension by bending up some of the main reinforcing rods and anchoring them for their full tensile strength beyond the edge of support. He recommends that bends be made close to the supports for the upper bends and at quarter points for the lower bends in beams carrying uniform load.

For girders carrying beams, bends should be made under the beams.

For anchorage he recommends that the rod should extend 40 to 50 diameters beyond the point where it intersects a line drawn at 45° with the horizontal from the bottom of the beam at the face of the support.

He recommends that the stress in bent-up rods be assumed to be that obtained by multiplying the excess of shear over that taken by the concrete (at 40 or 50 lb. per sq. in.) by the secant of the inclination of the rod with the vertical.

Mr. Godfrey also dissents from all parts of the report relating to rodded columns, or columns having longitudinal rods without close spaced hooping, for the reason that he holds that such reinforcement has not shown itself to have any definite value in tests on columns, and that analysis fails to show that any definite value can be ascribed to it, when such analysis takes into account the necessity for toughness in all columns; he also believes that dependence on such reinforcement has led to much unsafe construction and many failures. He would recognize as reinforced-concrete columns only such columns as have in addition to the longitudional rods a complete system of close-spaced hooping. He recommends the standardization of hooped columns and suggests that columns be reinforced by a coil or hoops of round steel having a diameter one-fortieth of the external diameter of the column and eight upright rods wired to the same, the pitch of the coil being one-eighth of the column diameter. He would consider available for resisting compressive stress the entire area of the concrete of a circular column or an octagonal column, but no part of the longitudinal rods or hooping. In a square column only 83 per cent of the area of concrete would be considered available. The compression he would recommend in columns (for 2000 lb. concrete) would be

$$P=670-12\frac{l}{d}$$

where P = allowed compression in lb. per sq. in.
l = length of column in inches
d = diameter of column in inches"

E. J. MOORE, *Chairman*,
WILLIAM P. ANDERSON,
ERNEST ASHTON,
ROBERT W. BOYD,
T. L. CONDRON,
A. W. FRENCH,
EDWARD GODFREY,
CHARLES W. KILLAM,
ARTHUR R. LORD,
ANGUS B. MACMILLAN,
SANFORD E. THOMPSON.

DISCUSSION.

Mr. E. J. Moore.—In view of the discussion received by the committee asking for reasons why several sections of this regulation do not conform to the latest report of the Joint Committee, the committee would like to give some of the reasons for these differences. Mr. A. R. Lord will answer some of those questions and questions relating to flat slabs. Mr. Moore.

Mr. A. R. Lord.—I think possibly the best way to take this up so as to save time and to get at the gist of the matter would be to state some of the questions received in the various discussions and to give our answers to those questions. Mr. Lord.

One of the first questions is, "Why not adopt the Joint Committee report as a whole, simply changing its form as far as necessary to make it into a building code instead of a discussion on reinforced concrete?" Now, the first reason why we do not adopt the Joint Committee report as a whole is that we believe we have considered data which possibly the Joint Committee did not consider, which possibly did not come to its attention, and which, if it had considered it, would have resulted in somewhat different rulings. A second reason is that we believe that the Joint Committee report is somewhat too rigid in the specifications of the moment,—in the matter of distribution of the moment between positive and negative. For instance, in several other respects it does not give an engineer an opportunity to use his best knowledge in the design of flat slab construction. In the third place, it contains a large number of arbitrary rules. The committee feels that in so far as can be avoided, it should not put arbitrary rules on any type of design, because those rules, even though adopted with thought at the time, are rarely considered in their connection with other arbitrary rules or with the principal requirements of the ordinance; the result being, as has been the case, we find, in the Joint Committee report, that some of these arbitrary rules work out to the detriment of the design rather than to the advantage of it. The fourth reason is that we believe the report of the Joint Committee is too conservative from the standpoint of the moment adopted; it condemns every flat-slab building that has ever been erected in the United States, so far as I know; it condemns buildings in the City of Chicago which have been tested both by load tests and by scientific extensometer tests to twice their designed working load, notwithstanding that those buildings, under that load, showed average stresses less than the designed stresses. In other words, this Joint Committee report, instead of calling for a factor of safety of four, as we consider desirable in reinforced concrete, or a factor of safety of two and one-half to three, as we prescribe in steel construction, calls for a factor of safety of six or even more. The fifth reason that we depart from the Joint Committee report is that some of its recommendations are in the nature of discussion and not suitable for a building code.

Mr. Lord. Now the second question, taking up the details of the Joint Committee Code, is, "Why do we use 0.09 $WL_1 (l_2 - qc)^2$ instead of the coefficient 0.107 times the same expression?" Our coefficient is that of the Chicago flat-slab ruling and that coefficient has proved very successful in actual design. It is the most conservative coefficient used in any city in the United States, and the extensometer tests of flat-slab buildings in Chicago and elsewhere have shown that the stresses under that design and ruling are very reasonable indeed. We do not think that a properly conservative design requires us to use a higher coefficient than the Chicago flat-slab ruling.

The third question is, "Why do we permit an equal division of the moment between plus and minus?" Please note that we do not prescribe that, we simply permit it. Where you have a flat slab without a drop, the positive moment may be equal to or in excess of the negative moment, as shown by extensometer tests. It would seem proper, in view of the facts, that a designer be permitted, where required to use a flat slab without a drop, to design that in accordance with the test data available and not to force on him design coefficients which apply only to flat slabs with large drops. The Joint Committee's fixed distribution with $62\frac{1}{2}$ per cent negative and $37\frac{1}{2}$ per cent positive, applies very well to the tests of Chicago buildings, with large drops; although the drops of Chicago buildings actually tested are much deeper than the committee would permit in its report. In one building on which an extensometer test has been made, and in which the stresses were exceedingly high (that building was probably tested more severely than any other I know of, with stresses running from 40,000 to 50,000 lb. per sq. in. in the reinforcing steel), the actual distribution of moment between the positive and the negative, with four panels loaded in a square, was 60 per cent positive and 40 per cent negative. Our report simply permits a man to use equal positive and equal negative moments in cases where such a distribution would actually occur. The Joint Committee would not let him get anywhere near the actual facts of that test.

The fourth question is, "Why not use the Joint Committee division between the inner and outer sections—that is, the division of the positive moment?" We have found by applying the Joint Committee rule to actual cases of four-way flat-slab design, that it requires, as a minimum condition, at least 2.1 times as much steel area in the direct band as is used in the diagonal band. Now, so far as we know, no flat-slab building of the four-way type has ever been built with anything like that ratio of steel. We cannot find any data in the tests which would cause us to think that that ratio of steel areas between the direct and diagonal bands is desirable, and we, therefore, prefer to leave a division possible which will accord with the present practice, which we see no reason for changing or for prescribing a change in. If a man wants to change it, however, he is free to do so under our rule: if he does not want to change it, we do not force him to, as the Joint Committee ruling does.

The fifth question is, "Why do we not give a formula for thickness for the flat slab when no drop is used?" The Joint Committee has a formula for flat-slab thickness when the drop is used and another when the drop is not used. The reason we have not put in that formula is that we desire to

avoid arbitrary rulings wherever possible. We already have four limitations on absolute thickness where the drop is not used, and do not see any reason for adding a fifth. One limitation is that the thickness shall not be less than $\frac{1}{32}$ of a span for a floor; the second is in the moment requirement at support; the third is in the requirement for diagonal tension and the fourth is in the requirement for punching shear, and our requirements, especially under the third and fourth heading, are in excess of the formula the Joint Committee gives, with the exception of very light loads, and there the limitation of $\frac{1}{32}$ comes in to take the place of it, so that the Joint Committee formula is practically of no use. **Mr. Lord.**

The sixth question I have is, "Why do we use 0.9 of the long side of a rectangular panel in determining the slab thickness rather than the entire length of the long side?" That also is a matter in which we have followed our judgment. The Joint Committee says that for a panel 19 by 20 ft., only 5 per cent out of square, you may use the average span in determining the slab thickness, but if the panel is 18 by 20 ft., you must use a 20-ft. span in determining the slab thickness; in other words, your slab thickness will be greater with an 18 by 20-ft. panel than a 19 by 20-ft. We have applied this 0.9 rule which prevents an undue decrease in slab thickness when the departure from the square panel becomes excessive. If you take a 16 by 20-ft. panel, which is quite rectangular, you will find that the moment has decreased in the long direct band by a greater amount than we permit the slab thickness to decrease; in other words, your moment has decreased more than 10 per cent, when we permit a decrease in the slab thickness of only 10 per cent.

The seventh question is, "Why do we not state a limit on drop thickness?" The Joint Committee says that the drop thickness shall not be more than one-half the slab thickness. The reason is that the tests of buildings which have thus far been made have nearly all been made on buildings with much deeper drop than that. The Joint Committee also specifies a limit on thickness of slab with a drop; for instance, you might take a design, as we have done, where the slab thickness figures out, by the formula, 9 in. They say that is a safe slab thickness to use, and that is correct. If you then figure the moment at the support, you find that you require a drop 14 in. thick, but you are limited to a drop which is only half as thick as the slab; you must therefore add $1\frac{1}{2}$ in. to your slab thickness to come within that arbitrary rule. A 9-in. slab is all right but you must make it $10\frac{1}{2}$ in. to satisfy this arbitrary rule of one-half the slab thickness. Then you have increased the dead weight of your slab and you must start all over on your computations. But the real reason for permitting a deeper drop is that Chicago buildings, where our recommended limitation is in force, have proved in every way satisfactory.

The eighth question is, "Why do we prescribe column moment of 0.022 $WL_1\,(l_2-qc)^2$?" This is simply a tentative value we have adopted. Other flat-slab rulings do not have a definite value for the column moment as compared with this coefficient of 0.022. We found in the test of the Franks building a coefficient of actual bending, including stresses in the concrete,

Mr. Lord. of 0.0138. I have used to quite a large extent the coefficient of 0.0185 with very satisfactory results. Our recommended column moment is nearly twice as large as the test showed, and considerably larger than I have used in design practice, and other members of the committee checked up. We understand and have provided that when a designer uses a large, stiff column, he must provide for a bigger moment than this. This is simply a minimum requirement and it is so stated in our ruling, but for a minimum applied to columns in the upper stories of buildings, it is adequate so far as our test data and experience goes.

The ninth question which I have is, "Why permit 50 per cent of the bars in a band to be lapped with a short lap in sections of high tensile stress, as, for instance, over the column capital?" There, again, we are taking the position that what has proved good in past experience should not be discarded or ruled out unless there is a good reason therefor. We have not been able to find any good reason for changing current practice. Everyone of the tests that have been made, including one with load on for a full year, had short laps in the bars over the column capital, and some had more of the rods lapped than the 50 per cent maximum which we have prescribed. Further, we have prescribed a lap of 80 diameters for such splices, which is much more than the bond would require.

The tenth and last question is, "Why permit an increase in compressive stress adjacent to the column capitals?" There we simply take the ground that this increase in the stress is permitted in continuous girders adjacent to supports, and we see no reason why, in the flat slab which is restrained in all directions, a similar increase in the compressive stress, should not be permitted.

Professor Talbot. PROF. ARTHUR N. TALBOT.—I desire to call attention to several matters in the report of the committee. These include in the treatment of the flat slab (*a*) the provision for the proportion which the negative moment may bear to the sum of the positive and negative moments, (*b*) the thickness of slab permitted by the minimum negative moment, (*c*) the allowable compressive stress in the column-head section, (*d*) the use of average span for determining thickness of slab in oblong panels, (*e*) the bending moment coefficients, (*f*) specification for magnitude of bending moment to be taken by columns, and (*g*) in the general matter on columns with longitudinal and lateral reinforcement, the working stress allowed for spiral reinforcement.

(*a*) *The Provision for the Proportion which the Negative Moment bears to the Sum of the Positive and Negative Moments.*—The report specifies as a minimum that the negative moment in a slab without a dropped panel may be taken as only one-half the specified numerical sum of negative moment and positive moment—40 per cent of it in the column-head section and 10 per cent in the midsection. Analysis definitely shows that for the homogeneous slab of uniform thickness, uniformly loaded and indefinite in extent, or having columns of sufficient stiffness, the magnitude of the negative moment in the column-head section and the midsection together is twice the magnitude of the positive moment in the inner section and the outer section. In other words, the total negative moment is two-thirds of the numerical sum of the

total negative moment and the total positive moment. Any question as to how the shear travels to the support will affect the distribution of the negative moment (*i. e.*, its intensity over the column-head section and midsection), the more directly the travel the greater the intensity of the negative moment in the column-head section—but the total magnitude of the negative moment may not be expected to be affected by this. To give some added strength and added stiffness when only a row of panels is loaded, it has been the practice to increase the provision for the positive moment somewhat. A very usual division considers the total bending moment to be made up of five-eighths as negative moment and three-eighths as positive moment. The proportion of negative moment here is fifteen-sixteenths of the full analytical value. The proportion permitted in the report is only 75 per cent of the full proportion which must come upon the section of negative moment. It would seem to me, that this small proportion should not be used, especially in connection with a 72 per cent bending moment coefficient.

(*b*) *The Thickness of Slab Permitted by Minimum Negative Moment.*—For a slab without dropped panel, using 16,000 lb. per sq. in. and 825 lb. per sq. in., and a column capital of 0.225 L, and considering 40 per cent of the specified moment as taken by the column-head section, the thickness of slab required to meet the condition of negative bending moment, in inches, becomes $t = 0.0194\ L\sqrt{w} + a$, where a is the distance from center of gravity of reinforcement to face of slab. This is from $d^2 = \dfrac{0.40 \times .0648\ wL^3 \times 12}{12 \times \frac{1}{2} L \times 140}$. This thickness is slightly more than the value specified for thickness of slab at center.

For comparison the thickness required to fulfil the regulations for flat slabs of the Chicago Building Department may be cited. Using 18,000 lb. per sq. in. and 700 lb. per sq. in., the minimum thickness for a slab without dropped panel is $t = 0.0245\ L\sqrt{w} + a$. This is from $d^2 = \dfrac{0.0333\ wL^3 \times 12}{12 \times \frac{1}{2} L \times 121}$. The effective thickness required by the Chicago Building Regulations is thus 25 per cent more than that which may be used by the regulations reported by the committee. It would seem to me that the limit of thickness due to negative bending moment should not be much below that of the Chicago Building Regulations.

In this connection, it may not be out of place to state the values for a slab with dropped panel of 0.3L. For the committee report this becomes

$$t = 0.0284\ L\sqrt{w} + a$$

For the Chicago Regulations it is

$$t = 0.0316\ L\sqrt{w} + a$$

(*c*) *The Allowable Compressive Stress at the Column-head Section.*—The report permits an increase of 10 per cent in the allowable working stress in compression at the column-head section. It should be noted that the value of the compressive stress which would be found by the formulas given in the

Professor Talbot. report is the average stress over the given section, and it is well known that there is a considerable variation in stress over a section, particularly in the case of the column-head section. Observations and tests indicate that the maximum stress in the column-head section may be expected to be 25 per cent more than the average stress in the column-head section or even higher. It would appear that this excess is far more than any gain which may be expected in the strength of concrete by reason of the concrete being stressed in two directions (and available tests indicate that any such gain is quite problematical) and that the use of average stress over the column-head section of itself is as large an allowance for favorable conditions of concrete as may properly be made. The condition is quite different from the allowance at the negative section made in the case of the beam or girder, for in that case the total moment provided is 1.33 times the total analytical moment, $[(\frac{1}{12}+\frac{1}{12})\div\frac{1}{8}=1.33]$, while here as the provision of the committee stands the total moment provided is only 72 per cent of the total analytical value, and the negative moment provided is only 54 per cent of the full negative moment. As the provision for compressive stress without this addition is higher than the usual practice, it would appear that this further allowance should not be accepted. The high compression stress permitted is the principal element in giving thinness of slab in region of negative moment.

(*d*) *The Use of Average Panel Length for Determining Thickness of Slab in Oblong Panels.*—There would seem to be little objection to the use of average panel length in oblong panels whose ratio is not less than 0.8 for determining the thickness in regions of positive moment, since the provision of the formula seems ample within this limit. However, this should not be a warrant for the use of average panel length in determining the thickness of slab at sections of negative moment, and there appears to be no analytical basis for using the average panel length instead of the longer panel length.

(*e*) *The Bending Moment Coefficient.*—The value of the total bending moment recommended by the Committee ($0.09\ wl_1\ (l_2-qc)^2$) is 72 per cent of bending moment of the external load as derived from analysis. As the full bending moment due to external load will be taken by the structure, any reduction made in the bending moment must be justified on the ground of permitting higher working stresses in the flat slab than are permitted for other forms of construction; that is, instead of increasing the allowable working stresses, the same result is accomplished by using the stresses specified for other forms of construction and allowing a reduction in the bending moment. It is evident that the relatively large breadth of structure in the flat slab, the large number of rods used, and other features of construction make the effect of local variations in the construction less than would be the case for narrow members like beams. How much the working stresses may be increased or how much the bending moment may be decreased is a matter which should be considered with great care. To my mind the use of 72 per cent of the analytical value (an increase in the working stresses of about 40 per cent) is not entirely warranted by known facts, even if a different division between the negative and the positive moment be made. As the provisions for proportion for negative moment and for the value in working stress in

Professor Talbot.

compression stand in the report, requiring only 54 per cent of the analytical value for the negative moment and allowing such high compressive stresses, the recommendations have no ground for acceptance. The Chicago Building Regulations give a bending moment of about 75 per cent of the analytical value or above. The 72 per cent bending moment is not so objectionable if it were not coupled with the provisions for low negative moment and high compressive stresses.

(*f*) *Specifications for Magnitude of Bending Moment to be Taken by Columns.*—The report provides that the resistance to bending is not to be taken as less than $0.022\ w_1l_1\ (l_2-qc)^2$. For full restraint, the analytical value of the negative bending moment for a panel width is $\frac{1}{12}\ w_1l_1(l_2-qc)^2$. The part taken by the columns may be two or more times as great as the provision specified in the report. It would seem better not to attempt to specify a value, for the amount taken by the columns will vary greatly with the relative thickness and length of slab and column.

(*g*) *The Working Stress Allowed for Spiral Reinforcement in Columns with Longitudinal and Lateral Reinforcement.*—The report makes a rather startling provision in allowing the effect of lateral reinforcement to be five times as much as the longitudinal reinforcement. The largest ratio in building regulations and in recommendations of engineers heretofore given to my knowledge is $2\frac{1}{2}$. Of the available tests known to me, a value of $2\frac{1}{2}$ or 3 is as great as is indicated by most tests and in many cases the results are considerably lower. While the question of the action of the hooped column and of what allowance should be made in working stresses is still unsettled, it would appear that this recommendation is very radical.

The general method of treatment of the flat slab followed in this report and in the report of the Joint Committee on Concrete and Reinforced Concrete is, I believe, an advance over previous methods of treatment, and its acceptance by engineers will tend towards more uniform methods of thinking about the flat slab and its action even if there are differences in coefficients and in various provisions.

I feel that from the activity of concrete constructors and the concrete interests any report which is adopted by the American Concrete Institute will be brought closely to the attention of building departments, architects and engineers. It seems important, therefore, that the Concrete Institute do not send out specifications which will not bear the scrutiny of wise engineering judgment.

Mr. Condron.

MR. T. L. CONDRON.—The attached table shows a direct comparison of the recommendations of the Joint Committee basis for designing flat-slab floor construction, with the proposed Standard Building Regulations of the American Concrete Institute, the Ruling of the Chicago Building Department, and the Akme Design Standards of the Condron Co.

From this table it is apparent that the proposed "Regulations" are practically identical with the Chicago ruling, as far as total moment is concerned, while the Joint Committee's recommendation calls for a moment 19 per cent above Chicago ruling. Considering the difference in allowable

Mr. Condron. stresses, the Joint Committee's recommendations require 35 per cent greater strength than the Chicago ruling.

The Akme Standards for "two-way" are about 10 per cent above the Chicago ruling for "four-way," using a drop panel and the same unit stresses.

Practically all of the vast amount of flat-slab floor construction throughout the United States has been built on designs that will meet or are less conservative than the so-called Chicago ruling. No one can deny that the majority of these structures, and certainly all of them that have been so

COMPARISON OF DESIGNING REGULATIONS FOR FLAT-SLAB FLOOR CONSTRUCTION.

		Coefficient. See Note.	Total. — Per Cent.	Total. + Per Cent.	Total Sum.	Column Head Section — Per Cent.	Middle Section — Per Cent.	Outer Section + Per Cent.	Inner Section + Per Cent.	Sum.	
Joint Committee.	Drop.	.107	62.5	37.5	100	50.0	12.5	28.1 to	9.4 to	100	
						50.0	12.5	22.5	15.0	100	
	No Drop	.107	62.5	37.5	100	50.0 to	12.5 to	28.1 to	9.4 to	100	
		.107	62.5	37.5	100	40.6	21.9	20.6	16.9	100	
Concrete Institute.	Drop.	.090	70.0 to	30.0 to	100	60.0 to	10.0 to	18.0 to	12.0 to	100	
		.090	60.0	40.0	100	50.0	20.0	28.0	22.0		Not over 100.
	No Drop.	.090	70.0 to	30.0 to	100	60.0 to	10.0 to	18.0 to	12.0 to	100	
			50.0	50.0	100	40.0	30.0	38.0	32.0		Not over 100.
Chicago Ruling.	4 way.	.087	66.7	33.3	100	53.2	13.4	20.0	13.4	100	Based on Head = .225 l.
	2 way.	.093	62.5	37.5	100	50.0	12.5	25.0	12.5	100	
Akme Standards 2 way.	Drop.	.096	60.0	40.0	100	47.0	13.0	27.0	13.0	100	
	No Drop.	.099	57.5	42.5	100	45.0	12.5	30.0	12.5	100	

NOTE.—The equivalent coefficient (a) in the expression Total Moment $= a\,w\,l\,(l - \frac{2}{3}c)^2$ when $c =$ diameter round head.

designed as to meet the Chicago ruling, or the "Akme Standards" are of ample strength to safely carry the loads for which they are intended.

It seems, therefore, that the rules included in Standard Building Regulations offered by the Committee should be adopted in the interests of the industry represented by the American Concrete Institute.

Professor Hatt. PROF. W. K. HATT.—I want to say a few words concerning this flat-slab controversy. As a member of this Joint Committee, I felt that any conclusion to which we came ought to be accompanied by some sort of a record as to the steps we took in coming to that conclusion. Any regulation or any design standards ought to submit themselves to a keen and logical analysis from the standpoint of mechanics and from the standpoint of all the informa-

tion in possession of the committee. I believe that the provisions of the Joint Committee are based upon that method of procedure. I realize also that evidence is evidence and that we may arrive at a correct conclusion not only from the standpoint of abstract logic but from experience in service of any material or any construction, and it seems to me that if this committee wishes to cut down standards which are arrived at by a process of logical thinking and critical examination, they ought to bring forth the evidence upon which they base their conclusions. I believe there is such evidence and that such evidence is accumulating. **Professor Hatt.**

We have in rebuttal to the various representations of the exhibited strength of slabs in service, certain elements as, for instance, that only a portion of a floor is loaded. There is a very large portion of the floor which is sharing in the resistance that is not under load. We have our old friend, the doctrine of the non-existence of tensile stresses which, like many doctrines, is very defective as a reflection of actual facts, and then we have the bending strength of columns and a number of things which make one cautious in accepting the results of tests. We ought to have a series of tests in which all these factors are considered. Then we can get up our conclusions and state to the on-coming generation just why the coefficients that we have adopted have been adopted. I think there is a matter of conservation here. There is no use of requiring a factor of safety of six or eight, when a factor of safety of four is enough.

PROPOSED STANDARD BUILDING REGULATIONS FOR THE USE OF REINFORCED CONCRETE.*

I. General.

Definition of Reinforced Concrete.

1. The term "Reinforced Concrete," as used in these Regulations, shall mean an approved mixture of portland cement with water and aggregates in which metal (generally steel) has been embedded in proportionately small sections, in such a manner that the metal and the concrete assist each other in taking stress.

Use.

2. Reinforced concrete may be used for all classes of buildings if the design is in accordance with good engineering practice and stresses are figured as indicated in these regulations.

Height of Buildings.

3. There shall be no limit upon the height of buildings of reinforced concrete except as limited, by general height restrictions for all types of buildings or by the strength requirements in these Regulations.

Permits.

4. Before permission is granted by the Building Department to erect any reinforced concrete building, complete general plans accompanied by specifications signed by the engineer or architect must be filed with the Building Department. Sufficient details shall be included in the plans submitted to make clear the exact dimensions and construction of the reinforced concrete portions of the building and the arrangement of the reinforcement so as to permit computation of all stresses. Specifications shall state the qualities and proportions of the materials to be used.

Copies of approved plans and specifications must be left on file with the Building Department for public inspection until the building is completed.

Inspection.

5. The construction of the building shall be inspected in detail by a representative of the architect or engineer who will keep a complete record of the progress of the work, including dates of placing concrete and dates of removing of forms. He shall also check the materials used, and the placing of same in the different parts of the building. These records shall be available for inspection by the Building Department.

Load Tests.

6. The Building Department may require the owner to make load tests on portions of the finished structure where there is a reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired, or where there is any doubt as to the sufficiency of the design. The test shall show that, with a load of twice the designed live load, the permanent deflection seven days after load is removed to be not more than 20 per cent of the total deflection under the test load. Load tests shall not be made before the concrete has been in place sixty days.

Posting of Floors.

7. The Building Department shall issue signed certificates to be posted on each floor of the building stating the safe carrying capacity per square foot.

* Presented to the Annual Convention, Feb., 1917, by the Committee on Reinforced Concrete and Building Laws, and, on motion of the Chairman of the Committee, action deferred for one year.

II. MATERIALS.

8. Only portland cement shall be used in reinforced concrete structures. Cement shall meet the requirements of the Standard Specifications for Cement of the American Society for Testing Materials as in effect at the time of the adoption of this regulation. (Standard 1, Am. Conc. Inst.) **Specifications, Cement.**

9. All cement used shall be tested and record of such tests shall be kept at the building site for inspection by the Building Department. No cement which has not met the requirements of the above specifications shall be used without the written approval of the Building Department. **Tests of Cement.**

10. All aggregates shall be of clean material, free from dust, soft particles, lumps of clay, vegetable loam, and all organic matter. **Aggregates—General.**

11. Fine aggregate shall consist of sand, or the screenings of gravel or crushed stone, graded from fine to coarse, and passing, when dry, a screen having $\frac{1}{4}$-in. diameter holes, it preferably shall be of siliceous material, and not more than 30 per cent by weight shall pass a sieve having 50 meshes per linear inch. **Fine Aggregates.**

12. Mortars composed of one (1) part portland cement and three(3) parts of fine aggregate by weight, when made into briquettes or into prisms or cylinders, should show a tensile or a compressive strength at least equal to the strength of 1 to 3 mortar of the same consistency, made with the same cement and standard Ottawa sand. If of lower strength, the proportion of cement shall be increased, but if less than 70 per cent, the fine aggregate shall be rejected. The amount of this cement shall be sufficient so that the compression strength of concrete test specimens made of the materials to be used shall show the values required. **Test of Fine Aggregates.**

13. Coarse aggregates shall consist of crushed stone, gravel, or slag, which is retained on a screen having $\frac{1}{4}$-in. diameter holes and graded in size from small to large particles. The maximum size of the coarse aggregate shall be such that the concrete will flow freely around the reinforcement. Bank gravel shall be separated from the sand before mixing. Slag shall be clean, dense, air cooled, blast furnace slag, weighing not less than 70 lb. per cu. ft. when loosely placed in the measure and containing not more than 1.3 per cent of sulphur as sulphides. **Coarse Aggregates.**

14. Cinders shall not be used as coarse aggregate in concrete for reinforced concrete structures without tests acceptable to the Building Department showing the strength of such concrete. Cinder concrete may be used for fireproofing, for floor and roof slabs, and for partitions. Where cinders are used as the coarse aggregate they shall be composed of hard, clean, vitreous clinker; free from sulphides, unburned coal or ashes. **Cinders.**

15. The water used in mixing concrete shall be free from oil, acid, alkalies or organic matter. **Water.**

16. Steel for reinforcement of concrete shall conform to the requirements of the specifications of the American Society for Testing Materials for Concrete Reinforcement Bars, as in effect at the time of the adoption of this regulation. **Reinforcement.**

Cold drawn steel wire made from billets may be used in floor and roof slabs, column hooping, and for temperature and shrinkage stresses. This

steel shall have an elastic limit between fifty thousand (50,000) and sixty-five thousand (65,000) lb. per sq. in. and an ultimate strength of not less than eighty-five thousand (85,000) lb. per sq. in.

All reinforcing steel shall be free from excessive rust, scale or coatings of any character which would tend to reduce or destroy the bond.

III. DETAILS OF CONSTRUCTION.

Forms. 17. Forms must be substantial and unyielding and sufficiently tight to prevent the leakage of mortar. Before placing concrete all forms shall be first thoroughly cleaned of all débris and preferably oiled to prevent adhesion of the concrete.

Preparation of Reinforcement. 18. All bars must be carefully bent as required by plans.

Placing of Reinforcement. 19. Reinforcement shall be accurately located in the forms and secured against displacement.

Steel Splices. 20. Where it is necessary to splice reinforcing steel, this shall be done, by providing a lap sufficient to transfer the stress between bars by bond and shear, or by a mechanical connection such as a screw coupling. Splices at point of maximum stress should be avoided.

Construction Joints. 21. Vertical fill lines between two fills of concrete must be selected so that the resulting joint will have the least possible effect upon the strength of the structure. Before making the second fill, the concrete previously placed shall be thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of one (1) part Portland cement and not more than two (2) parts fine aggregate.

Construction joints for columns should be made at underside of floor construction, haunches and column capitals being considered as part of the floor construction, and should be poured monolithically. Where reinforced concrete columns have flaring heads or where structural steel columns are used, concrete for slab and column heads may be poured immediately after the concrete for the column shaft.

In general, fill lines in floors should be selected near the center of spans of slabs, beams and girders. Where shear is present at the joint, adequate provision shall be made for resisting same by inclining joint or providing sufficient reinforcement.

Measuring Ingredients. 22. Methods of measuring of the various ingredients of concrete, including the water, shall be used which will secure separate and uniform measurements, of the proportions required. Measurements shall be made by volume; 94 lb. of cement to be considered as a cubic foot.

Mixing—General. 23. The ingredients of concrete shall be thoroughly mixed to the desired consistency and the mixing shall continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous.

Machine Mixing. 24. In mixing by machine a mixer of a type which insures the uniform distribution of the materials throughout the mass shall be used.

Hand Mixing. 25. When it is necessary to mix by hand, the mixing shall be done on a watertight platform, and all ingredients shall be turned together at least six times and until the resulting mass is homogeneous in appearance and color.

26. The materials must be mixed wet enough to produce a concrete of such a consistency that it will flow sluggishly into the forms and about the metal reinforcement, and at the same time can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar. **Consistency.**

27. Mortar or concrete shall not be re-mixed with water and used after it has partly set. **Re-Tempering.**

28. Concrete after the completion of the mixing shall be transported as rapidly as practicable from the place of mixing to the place of final deposit. The concrete shall be deposited in such a manner that it will flow sluggishly around the steel reinforcement and shall be rammed or agitated by suitable tools in such a manner as to produce thoroughly compact concrete. **Placing of Concrete.**

29. Concrete shall not be placed in water unless unavoidable; if necessary to do this excessive mixing water shall be used to prevent the cement from being separated from the aggregate. **Placing in Water.**

30. The concrete at the end of each fill shall be cleaned of laitance or other deleterious material which would detract from the quality of the concrete. After forms are removed, any porous sections of concrete shall be cleaned out and filled in a manner to meet the approval of the Building Department. **Finishing.**

31. The face of concrete exposed to rapid drying shall be kept damp for a period of at least five days. **Protection in Warm Weather.**

32. Concrete shall not be mixed or deposited unless it is maintained at a temperature not less than 50 deg. F. during mixing, placing, and for at least 72 hours thereafter, or until the concrete has thoroughly hardened. **Protection in Cold Weather.**

33. Under no consideration shall forms be removed until the concrete has hardened sufficiently to permit their removal with safety. **Removal of Forms.**

Where there is danger of frozen concrete being mistaken for properly hardened concrete, heat shall be applied before tests for hardness are made.

34. As soon as a section of form is removed, shoring shall be provided as necessary to carry the weight of the new concrete and other loads brought upon the construction in acting as a support for upper floors. Careful consideration must be given to the loads carried and the strength of the new concrete before any shoring is removed. **Temporary Supports.**

IV. DESIGN.

35. All reinforced concrete construction shall be designed to meet the conditions of loading (including bending in columns) without stressing the materials used beyond the safe working stresses specified. **Conditions.**

36. The dead loads shall be the weight of the permanent structure. The weight of reinforced stone, gravel or slag concrete shall be taken as 144 lb. per cu. ft.; the weight of cinder concrete as 100 lb. per cu. ft. **Dead Loads.**

37. The live load shall be the working or variable load for which the structure is designed. **Live Loads.**

38. All parts of a structure shall be designed to carry safely the entire combined dead and live loads with the exception that the loads on columns and foundations may be reduced by considering that columns in top story carry the total live and dead load above them; columns in next to top story **Reduction of Loads.**

carry the total dead load and eighty-five (85) per cent of the total live load above; columns in the next lower story, the total dead load and eighty (80) per cent of the total live load above; and thus on downward, reducing at each story the percentage of total live loads carried, by 5, until a reduction of fifty (50) per cent is reached. The columns in this and in every story below this point, shall be proportioned to carry the total dead load and at least fifty (50) per cent of the total live load of all the floors and roofs above them.

For warehouses the increment of reduction per story shall be $2\frac{1}{2}$ per cent instead of 5 per cent.

General Assumptions.

39. As a basis for calculations for the strength of reinforced concrete construction the following assumptions shall be made:

(*a*) Calculations shall be made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

(*b*) A plane section before bending remains plane after bending.

(*c*) The modulus of elasticity of concrete in compression within the usual limits of working stresses is constant.

(*d*) No allowance shall be made for the tensile value of concrete.

(*e*) Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses the two materials will, therefore, be stressed in proportion to their moduli of elasticity.

(*f*) The ratio of the modulus of elasticity of steel to that of concrete shall be taken as follows:

1. One-fortieth that of steel when the strength of the concrete is taken as not more than eight hundred (800) lb. per sq. in.

2. One-fifteenth that of steel when the strength of the concrete is taken as greater than eight hundred (800) lb. per sq. in. or less than twenty-two hundred (2200) lb. per sq. in.

3. One-twelfth that of steel when the strength of the concrete is taken as greater than twenty-two hundred (2200) lb. per sq. in. or less than twenty-nine hundred (2900) lb. per sq. in.

4. One-tenth that of steel when the strength of the concrete is taken as greater than twenty-nine hundred (2900) lb. per sq. in.

Strength of Materials.

40. The ultimate strength of concrete shall be that developed at an age of 28 days in cylinders 8 in. in diameter and 16 in. in length or 6 in. in diameter and 12 in. in length of the consistency and proportions to be used in the work, made and stored under laboratory conditions, but in no case shall the values exceed those allowed for granite in the table below. In the absence of definite knowledge in advance of construction as to just what strength may be developed the following values may be used:

TABLE OF STRENGTHS OF DIFFERENT MIXTURES OF CONCRETE.

(In Pounds per Square Inch.)

Aggregate	1 : 3	1 : 4½	1 : 6	1 : 7½	1 : 9
Granite, trap rock	3300	2800	2200	1800	1400
Gravel, hard limestone, hard sandstone, and approved slag	3000	2500	2000	1600	1300
Soft limestone and sandstone	2200	1800	1500	1200	1000
Cinders	800	700	600	500	400

Safe Working Stresses.

41. Reinforced concrete structures shall be so designed that the stresses, figured in accordance with these regulations, in pounds per square inch, shall not exceed the following:

Extreme fiber stress in concrete in compression—$37\frac{1}{2}$ per cent of the compressive strength specified in Section 40. Adjacent to the support of continuous members, 41 per cent, provided the member frames into a mass of concrete projecting at least 50 per cent of the least dimension of the member on all sides of the compression area of the member.

Concrete in direct compression—25 per cent of the compressive strength specified in Section 40.

Shearing stress in concrete when main steel is not bent and when steel is not provided to resist diagonal tension—2 per cent of the compressive strength specified in Section 40.

Punching shear in concrete—$7\frac{1}{2}$ per cent of the compressive strength specified in Section 40.

Shearing stress in concrete when steel to assist in resisting diagonal tension is provided—$7\frac{1}{2}$ per cent of the compressive strength specified in Section 40 providing that sufficient web reinforcement is supplied to carry the stresses in excess of the value allowed for the unreinforced concrete; and providing further, that this web reinforcement extends from top to bottom of beam and is adequately anchored to the horizontal reinforcement. If main reinforcing bars are bent up and anchored, they may be considered as part of the web reinforcement.

Bond stress between concrete and plain reinforcing bars—4 per cent of the compressive strength.

Bond stress between concrete and approved deformed bars—5 per cent of the compressive strength.

Bearing upon a surface of concrete at least twice the loaded area—50 per cent of the compressive strength of the concrete.

Tensile stress in steel—16,000 lb. per sq. in., except that for steel having an elastic limit of at least 50,000 lb., a working stress of 18,000 lb. per sq. in. will be allowed.

Girder, Beam, and Slab Construction.

42. In determining the bending moment in slabs, beams and girders, the load carried by the member shall include both the dead and the live loads.

The span of the member shall be the distance center to center of supports but not to exceed the clear span plus the depth of the member, except that for continuous or fixed members framing into other reinforced concrete members the clear span may be used.

For continuous members supported upon brackets making an angle of not more than 45 deg. with the vertical, and having a width not less than the width of the member supported, the span shall be the clear distance between brackets plus one-half the total depth of the member.

If the brackets make a greater angle than 45 deg. with the vertical, only that portion of the bracket within the 45 deg. slope shall be considered.

For members uniformly loaded the bending moment shall be assumed as $\frac{WL}{F}$, where W=total load; L=span; and $F=8$ for members simply

supported, 10 for both negative and positive bending moment for members restrained at one end and simply supported or partially restrained at the other, and 12 for both negative and positive bending moment for members fixed or continuous at both supports. The above bending moments for continuous members apply only when adjacent spans are approximately equal.

A special condition of loading to be reduced to equivalent uniformly distributed loading in accordance with approved engineering practice. For members having one end simply supported or partially restrained, at least fifty (50) per cent of the tension reinforcement required at center of span shall be bent up and adequately anchored to take bending moment at exterior support.

Slabs.

43. The main tensile reinforcement shall not be farther apart than $2\frac{1}{2}$ times the thickness of the slab. For slabs designed to span one way, steel having an area of at least two-tenths of one per cent of section of slab shall be provided transverse to main reinforcement, and this transverse reinforcement shall be further increased in the top of the slab over girders to prevent cracking and the main steel in slabs parallel and adjacent to girders may be reduced accordingly. Where openings are left through slabs, extra reinforcement shall be provided to prevent local cracks developing. This reinforcement shall in no case be less than $\frac{1}{4}$ sq. in. in section and must be securely anchored at ends. Floor finish when placed monolithic may be considered part of the structural section.

T Beams and Girders.

44. Where adequate bond and shearing resistance between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall not exceed on either side of the beam one-sixth of the span length of the beam nor be greater than 6 times the thickness of the slab on either side of the beam, the measurements being taken from edge of web.

Web reinforcement for beams and girders shall be so designed as to adequately take up throughout their length all stresses not taken up by the concrete. Web members shall be spaced not to exceed three-fourths of the effective depth of the beam in that portion where the web stresses exceed the allowable value of concrete in shear. Web reinforcement, unless rigidly attached, shall be placed at right angles to the axis of the beam or girder, and carried around or securely anchored to longitudinal steel in both the tension and compression areas.

Tile and Joist Floors.

45. Wherever floors are built with a combination of tile or other fillers between reinforced concrete joists, the following rules regarding the dimensions and methods of calculations of construction shall be observed:

(*a*) Wherever a portion of the slab above the fillers is considered as acting as a T-beam section, the slab portion must be cast monolithic with the joist and have a minimum thickness of two (2) in.

(*b*) Wherever porous fillers are used which will absorb water from the concrete, care must be taken to thoroughly saturate same before concrete is placed.

(*c*) All regulations given above for beam and girder floors shall apply to tile and joist floors.

(*d*) The sections of fillers shall be together and reasonably tight before concrete is placed.

46. Continuous flat slab floors, reinforced with steel rods or mesh and supported on spaced columns in orderly arrangement, shall conform to the following requirements: **Flat Slab Floors.**

(*a*) *Notation and Nomenclature.*—In the formulæ let

w = total dead and live load in pounds per square foot of floors.

l_1 = span in feet center to center of columns parallel to sections on which moments are considered.

l_2 = span in feet center to center of columns perpendicular to sections at which moments are considered.

c = average diameter of column capital in feet at point where its thickness is $1\frac{1}{2}$ in.

q = distance from center line of the capital to the center of gravity of the periphery of the half capital divided by $\frac{1}{2}c$. For round capitals q may be considered as two-thirds and for square capitals as three-quarters.

t = total slab thickness in inches.

L = average span in feet center to center of columns, but not less than 0.9 of the greater span.

The column head section, mid section, outer section, and inner section' are located and dimensioned as shown in Fig. 1. Corresponding moments shall be figured on similar sections at right angles to those shown in Fig. 1.

(*b*) *Structural Variations.*—Flat slab floors may be built with or without caps, drops or paneled ceilings. These terms are illustrated in Fig. 2.

Where caps are employed they shall be considered a part of the columns and the column capital dimension c shall be found by extending the lines of the capital below to an intersection with the plane of the under surface of the slab as indicated in Fig. 2-b. The cap shall be large enough to inclose this extension of the capital lines.

The column capital profile shall not fall at any point inside an inverted cone drawn, as shown in Fig. 2-a, from the periphery of the designed capital of diameter c and with a base angle of 45 deg. The diameter of the designed capital c shall be taken where the vertical thickness of the column capital is at least $1\frac{1}{2}$ in.

The drop, where used, shall not be less than 0.3 of L in width.

Where paneled ceilings are used the paneling shall not exceed one-third of the slab thickness in depth and the dimension of the paneling shall not exceed 0.6 of the paneled dimension. (See Fig. 2-c.)

(*c*) *Slab Thickness.*—The slab thickness shall not be less than $t=0.02\,L\sqrt{w}+1$ in.

In no case shall the slab thickness be less than $\frac{1}{32}\,L$ for floor slabs nor less than $\frac{1}{40}\,L$ for roof slabs.

(*d*) *Design Moments.*—The numerical sum of the positive and negative moments in foot-pounds shall not be less than $0.09\,wl_1\,(l_2-qc)^2$. Of this total amount not less than 40 per cent shall be resisted in the column head

sections. Where a drop is used, not less than 50 per cent shall be resisted in the column head sections.

Of the total amount not less than 10 per cent shall be resisted in the mid section.

Of the total amount not less than 18 per cent shall be resisted in the outer sections.

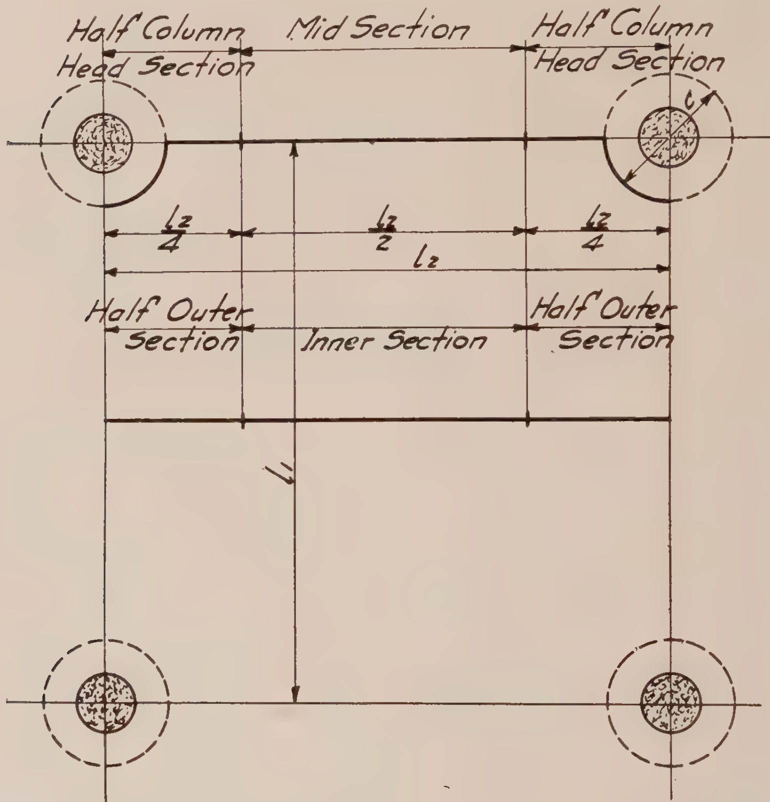


FIG. 1.

Of the total amount not less than 12 per cent shall be resisted on the inner sections.

(e) *Exterior Panels.*—The negative moments at the first interior row of columns and the positive moments at the center of the exterior panel on sections parallel to the wall, shall be increased 20 per cent over those specified above for interior panels. If girders are not provided along the column line, the reinforcement parallel to the wall for negative moment in the column

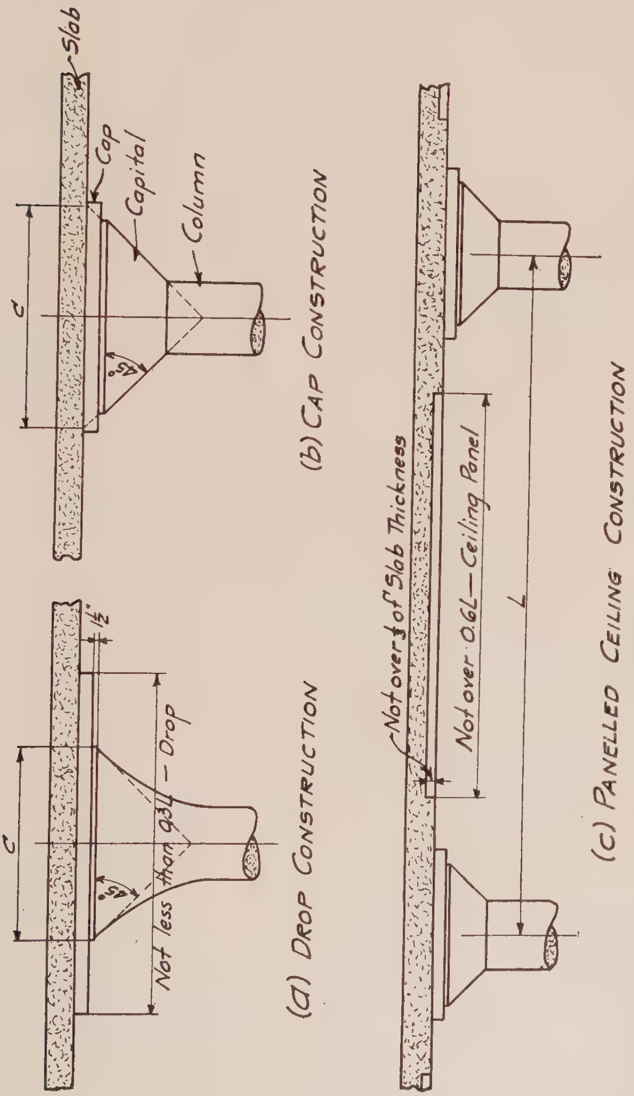

Slab
Cap
Capital
Column
c
45°
(b) CAP CONSTRUCTION
c
45°
1½"
Not less than 0.3L — Drop
(a) DROP CONSTRUCTION
Not over ⅓ of Slab Thickness
Not over 0.6L—Ceiling Panel
L
(c) PANELLED CEILING CONSTRUCTION

FIG. 2.

head section and for positive moment in the outer section adjacent to the wall, shall be altered in accordance with the change in the value of c. The negative moment on sections at the wall and parallel thereto should be determined by the conditions of restraint, but must never be taken less than 50 per cent of those for the interior panels.

(*f*) *Reinforcement.*—In the calculation of moments all the reinforcing bars which cross the section under consideration and which fulfil the requirements given under "Arrangement of Reinforcement" may be used. For a column head section reinforcing bars parallel to the straight portion of the section do not contribute to the negative resisting moment for the column head section in question. The sectional area of bars, crossing the section at an angle, multiplied by the sine of the angle between these bars and the straight portion of the section under consideration may be taken to act as reinforcement in a rectangular direction.

(*g*) *Point of Inflection.*—For the purpose of making calculations of moment at sections away from the sections of negative moment and positive moment already specified, the point of inflection shall be taken at a distance from center line of columns equal to $\frac{1}{5}(l_2-qc)+\frac{1}{2}\ qc$. This becomes $\frac{1}{5}(l_2+c)$ where capital is circular. For slabs having drop panels the coefficient of $\frac{1}{4}$ should be used instead of $\frac{1}{5}$.

(*h*) *Arrangement of Reinforcement.*—The design should include adequate provision for securing the reinforcement in place so as to take not only the maximum moments but the moments of intermediate sections. If bars are extended beyond the column capital and are used to take the bending moment on the opposite side of the column, they must extend to the point of inflection. Bars in diagonal bands used as reinforcement for negative moment should extend on each side of the line drawn through the column center at right angles to the direction of the band a distance equal to 0.35 of the panel length, and bars in the diagonal bands used as reinforcement for positive moment, should extend on each side of the diagonal through the center of the panel a distance equal to 0.35 of the panel length. Bars spliced by lapping and counted as only one bar in tension shall be lapped not less than 80 diameters if splice is made at point of maximum stress and not more than 50 per cent of the rods shall be so spliced at any point in any single band or in any single region of tensile stress. Continuous bars should not all be bent up at the same point of their length, but the zone in which this bending occurs should extend on each side of the assumed point of inflection.

(*i*) *Tensile and Compressive Stresses.*—The usual method of calculating the tensile and compressive stresses in the concrete and in the reinforcement, based on the assumptions for internal stresses, should be followed. In the case of the drop panel, the section of the slab and drop panel may be considered to act integrally for a width equal to the width of the column head section. Within the column head section the allowable compression may be increased as prescribed in Section 41 for continuous members.

(*j*) *Provision for Diagonal Tension and Shear.*—In calculations for the shearing stress which is to be used as the means of measuring the resistance to diagonal tension stress, it shall be assumed that the total vertical shear

on a column head section constituting a width equal to one-half the lateral dimension of the panel, for use in determining critical shearing stresses, shall be considered to be one-fourth of the total dead and live load on a panel for a slab of uniform thickness, and to be 0.3 of the sum of the dead and live loads on a panel for a slab with drop panels. The formula for shearing unit stress shall be $v = \frac{0.25W}{bjd}$ for slabs of uniform thickness and $v = \frac{0.30W}{bjd}$ for slabs with drop panels, where W is the sum of the dead and live load on a panel, b is half the lateral dimension of the panel measured from center to center of columns, and jd is the lever arm of the resisting couple at the section.

The calculation for punching shear shall be made on the assumption of a uniform distribution over the section of the slab around the periphery of the column capital and also of a uniform distribution over the section of the slab around the periphery of the drop panel, using in each case an amount of vertical shear greater by 25 per cent than the total vertical shear on the section under consideration.

The values of working stresses should be those recommended for diagonal tension and shear in Section 41.

(*k*) *Walls and Openings.*—Girders or beams shall be constructed to carry walls and other concentrated loads which are in excess of the working capacity of the slab. Beams should also be provided in case openings in the floor reduce the working strength of the slab below the required carrying capacity.

(*l*) *Unusual Panels.*—The coefficients, steel distribution, and thicknesses recommended are for slabs which have three or more rows of panels in each direction and in which the sizes of the panels are approximately the same. For structures having a width of one or two panels and also for slabs having panels of markedly different sizes, an analysis should be made of the moments developed in both slab and columns and the values given herein modified accordingly.

(*m*) *Bending Moments in Columns.*—Provision shall be made in both wall columns and interior columns for the bending moment which will be developed by unequally loaded panels, eccentric loading, or uneven spacing of columns. The amount of moment to be taken by a column will depend on the relative stiffness of columns and slab, and computations may be made by rational methods such as the principle of least work or of slope and deflection. Generally the largest part of the unequalized negative moment will be transmitted to the columns and the columns should be designed to resist this bending moment. Especial attention should be given to wall columns and corner columns. Column capitals should be designed, and reinforced where necessary, with these conditions in mind.

The resistance of any column to bending in a direction parallel to l_2 shall not be less than $0.022\, w_1 l_1\, (l_2 - qc)^2$, in which w_1 is the designed live load per square foot. In determining the resistance to be provided in exterior columns in a direction perpendicular to the wall the full dead and live load w shall be used in the above formula in place of w_1. The moment in such exterior columns may be reduced by the balancing moment of the

weight of the structure which projects beyond the supporting wall column center line.

Where the column extends through the story above, the resisting moment shall be divided between the upper and the lower columns in proportion to their stiffness. The calculations of combined stresses due to bending and direct load shall not exceed by more than 50 per cent the stresses allowed for direct load.

Columns, General.

47. Reinforced concrete columns, for the working stresses hereinafter specified, shall have a gross width or diameter not less than one-fifteenth of the unsupported height nor less than twelve (12) in. All vertical reinforcement shall be secured against lateral displacement by steel ties not less than $\frac{1}{4}$ in. in diameter, placed not farther apart than 15 diameters of the vertical rods or more than 12 in.

Columns with Longitudinal Reinforcement.

48. For columns having not less than 0.5 per cent nor more than 6 per cent of vertical reinforcement, the allowable working unit stress for the net section of the concrete shall be 25 per cent of the compressive strength specified in Section 40, and the working unit stress for the steel shall be based upon the ratio of the moduli of elasticity of the concrete and steel. Concrete to a depth of 1 in. shall be considered as protective covering and not a part of the net section.

Columns with Longitudinal and Lateral Reinforcement.

49. Columns; having not less than 1 per cent nor more than 6 per cent of vertical reinforcement and not less than 0.5 per cent nor more than 2 per cent of lateral reinforcement in the form of hoops or spirals spaced not farther apart than one-sixth of the outside diameter of the hoops or spirals nor more than 3 in.; shall have an allowable working unit stress for the concrete within the outside diameter of the hoops or spirals equal to 25 per cent of the compressive strength of the concrete, as given in Section 40, and a working unit stress on the vertical reinforcement equal to the working value of the concrete multiplied by the ratio of the specified moduli of elasticity of the steel and concrete, and a working load for the hoops or spirals determined by considering the steel in hoops or spirals as five times as effective as longitudinal reinforcing steel of equal volume. The percentage of lateral reinforcement shall be taken as the volume of the hoops or spirals divided by the volume of the inclosed concrete in a unit length of column. The hoops or spirals shall be rigidly secured at each intersection to at least four (4) verticals to insure uniform spacing. The percentage of longitudinal reinforcement used shall be not less than the percentage of the lateral reinforcement.

Structural Steel and Concrete Columns.

50. For steel columns filled with concrete and encased in a shell of concrete at least 3 in. thick where the steel is calculated to carry the entire load, the allowable stress per square inch shall be determined by the following formula: $18{,}000-70\,\frac{L}{R}$ but shall not exceed 16,000 lb.—where L = unsupported length in inches and R = least radius of gyration of steel section in inches. The concrete shell shall be reinforced with wire mesh or hoops weighing at least 0.2 lb. per sq. ft.

Footings—General.

51. Symmetrical, concentric column footings shall be designed for punching shear, diagonal tension and bending moment.

52. The area effective to resist punching shear in column footings shall be considered as the area having a width equal to the perimeter of the column or pier and a depth equal to the depth of footing from top to center of reinforcing steel. **Punching Shear in Footings.**

53. Shearing stresses as indicative of diagonal tension shall be measured in footings on vertical sections distant from the face of the pier or column equal to the depth of the footing from top to center of reinforcing steel. **Diagonal Tension in Footings.**

54. The bending moment in isolated column footings at a section taken at edge of pier or column shall be determined by multiplying the load on the quarter footing (after deducting the quarter pier or column area) by six-tenths of the distance from the edge of pier or column to the edge of footing. The effective area of concrete and steel to resist bending moment shall be considered as that within a width extending both sides of pier or column, a distance equal to depth of footing plus one-half the remaining distance to edge of footing, except that reinforcing steel crossing the section other than at right angles, shall be considered to have an effective area determined by multiplying the sectional area by the sine of the angle between the bar and the plane of the section. **Bending Moment in Footings.**

55. In designing footings, careful consideration must be given to the bond stresses which will occur between the reinforcing steel and the concrete. **Bond Stresses in Footings.**

56. Walls shall be reinforced by steel rods running horizontally and vertically. Walls having an unsupported height not exceeding fifteen times the thickness may be figured the same as columns. Walls having an unsupported height not more than twenty-five times the thickness may be figured to carry safely a working stress of $12\frac{1}{2}$ per cent of the compressive strength specified in Section 40. **Walls—General.**

57. Exterior walls shall be designed to withstand wind loads or loads from backfill. The thickness of wall shall in no case be less than 4 in. **Exterior Walls.**

58. The reinforcement in columns and girders shall be protected by a minimum thickness of 2 in. of concrete; in beams and walls by a minimum of $1\frac{1}{2}$ in.; in floor slabs by a minimum of $\frac{3}{4}$ in.; in footings by a minimum of 3 in. **Protection.**

AMERICAN CONCRETE INSTITUTE.

STANDARD NO. 5.

ADOPTED BY LETTER BALLOT, APRIL 10, 1917.

STANDARD SPECIFICATIONS FOR ONE-COURSE CONCRETE HIGHWAY.

I. Materials.

1. *Cement.*—The cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement, adopted by the American Society for Testing Materials, September 1, 1916, with all subsequent amendments and additions thereto adopted by said Society and by this Institute (Standard No. 1).

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the engineer a fifty (50) pound sample of each of the fine and coarse aggregates proposed for use. The samples shall be tested and if found to pass the requirements of the specifications similar material shall be considered as acceptable for the work. Aggregates containing frost or lumps of frozen material shall not be used.

Fine Aggregate: Fine aggregate shall consist of natural sand or screenings from hard, tough, durable crushed rock or gravel, consisting of quartzite grains or other equally hard material graded from fine to coarse with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes per linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch, and not more than five (5) per cent shall pass a sieve having one hunderd (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other deleterious matter, nor more than three (3) per cent by weight of clay or loam. Routine field tests shall be made on fine aggregate as delivered. If there is more than seven (7) per cent of clay or loam by volume in one (1) hour's settlement after shaking in an excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement, and three (3) parts fine aggregate, by weight, when made into briquettes, shall show a tensile strength (at seven (7) and twenty-eight (28) days) equal to or greater than the strength of briquettes composed of one (1) part of the same cement and three (3) parts standard Ottawa

sand by weight. The percentage of water used in making the briquettes of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquettes of standard consistency. In other respects all briquettes shall be made in accordance with the methods outlined in the Standard Specifications and Tests for Portland Cement adopted by the American Society for Testing Materials, Sept. 1, 1916.

Coarse Aggregate: Coarse aggregate shall consist of clean, hard, tough, durable crushed rock or pebbles graded in size, free from vegetable or other deleterious matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall be such as to pass a two (2) inch round opening and shall range from two (2) inches down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

3. *Mixed Aggregate.*—Crusher run stone, bank run gravel or artificially prepared mixtures of fine and coarse aggregate shall not be used.

4. *Water.*—Water shall be clean, free from oil, acid, alkali or vegetable matter.

5. *Reinforcement.*—All reinforcement shall be free from excessive rust, scale, paint or coatings of any character which will tend to destroy the bond.

6. *Joint Filler.*—Joint filler shall consist of prepared strips of fiber matrix and bitumen, or similar material of approved quality, one-quarter ($\frac{1}{4}$) inch in thickness. Where the joints are protected with metal plates the joint filler shall be made to conform to the cross-section of the pavement, and where unprotected joints are used the width of the joint filler shall be at least one-half ($\frac{1}{2}$) inch greater than the thickness of the pavement at any point. Prior to submitting bid the contractor shall obtain approval of the engineer for the joint filler which he proposes to use.

7. *Joint Protection Plates.*—Soft steel plates for the protection of the edges of the concrete at transverse joints shall be not less than two and one-half ($2\frac{1}{2}$) inches in depth and not less than one-eighth ($\frac{1}{8}$) nor more than one-quarter ($\frac{1}{4}$) inch average thickness. The plates shall be of such form as to provide for rigid anchorage to the concrete. The type and method of installation of joint protection plates shall be approved by the engineer.

8. *Shoulders.*—(Materials for the construction of shoulders shall be here described as desired by the engineer).

II. GRADING.

9. *Defined.*—"Grading" shall include all cuts, fills, ditches, borrow pits, approaches and all earth or rock moving for whatever purpose where such work is an essential part of or necessary to the prosecution of the contract.

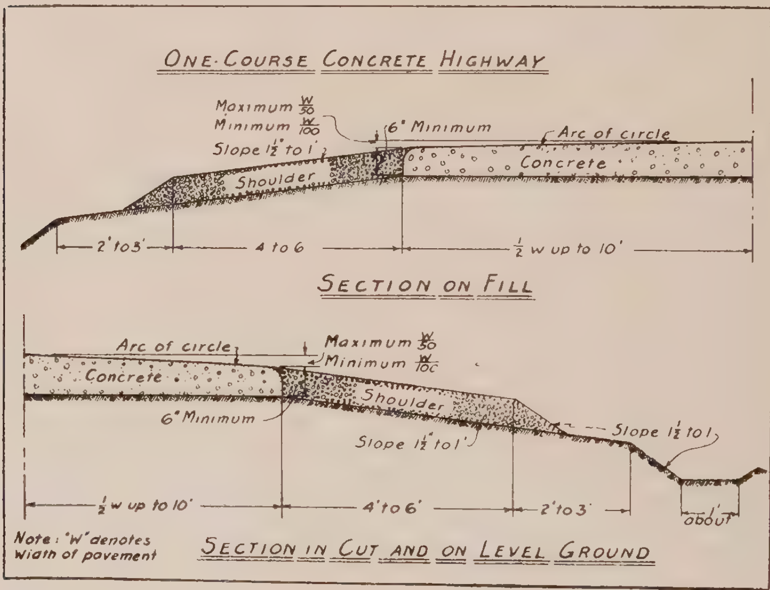
10. *Engineer's Stakes.*—Stakes will be set by the engineer for the center line, side of slopes, finished grade and other necessary points properly marked for the cut or fill. When the established grade is approached the final grade stakes will be set for which .... day's notice must be given to the engineer.

11. *Free Haul.*—Excess material shall be disposed of as directed by the engineer, the free haul not to exceed .... feet.

12. *Over-Haul.*—Materials hauled a greater distance than the free haul from the place of excavation shall be paid for at the rate of .... cents per cubic yard for each additional .... feet.

13. *Cuts and Fills.*—In cuts the final grade shall be obtained by rolling with a roller weighing not less than five (5) nor more than ten (10) tons. When a fill of one (1) foot or less is required to bring the surface to grade, all vegetable matter shall be removed before making the fill.

Embankments shall be formed of earth or other approved materials and shall be constructed in successive layers the first of which shall extend entirely across from the toe of the slope on one side to the toe of the slope on the other side and successive layers shall extend entirely across embankments from slope



to slope. Each layer, which shall not exceed one (1) foot in depth, shall be thoroughly rolled with a roller weighing not less than five (5) nor more than ten (10) tons. The roller shall pass over the entire area of each layer of the fill at least twice. The sides of the embankment shall be kept lower than the center during all stages of the work and the surface maintained in condition for adequate drainage. The use of muck, quicksand, soft clay or spongy material which will not consolidate under the roller is prohibited.

When the material excavated from the cuts is not sufficient to make the fills shown on the plans, the contractor shall furnish the necessary extra material to bring the fills to the proper width and grade. When the earth work is completed, the cross-section of the roadbed shall conform to the cross-sectional drawings and profile attached hereto.

14. *Slopes.*—All slopes must be properly dressed to lines given by the engineer.

III. DRAINAGE.

15. *Drainage.*—The contractor shall construct such drainage ditches as will insure perfect surface and subsurface drainage during construction, and such work shall be completed to the satisfaction of the engineer, prior to the preparation of the roadbed, as herein specified.

Tile drains shall be placed as shown in the drawings attached hereto. Tile to be laid in a trench at least .... (........) inches wide and .... (........) feet deep below the established grade of the finished road. Such trench shall be backfilled with crushed stone or pit run gravel, with fine material removed, which, after light tamping, shall be .... (........) inches in depth.

Open ditches must be constructed along the concrete road as shown on the attached drawing, the dimensions, side slopes and grade of said ditches being as shown on the cross-section drawings and profile attached hereto.

At the time of acceptance of the road or when concreting is discontinued for the winter, the ditches must be in finished condition with clean slopes and bottom containing no obstructions to the flow of water.

IV. SUBGRADE.

16. *Construction.*—The subgrade shall be brought to a firm density by rolling the entire area with a self-propelled roller. All portions of the surface of the subgrade which are inaccessible to the roller shall be thoroughly tamped with a hand tamp weighing not less than fifty (50) pounds, the face of which shall not exceed one hundred (100) square inches in area. All soft, spongy or yielding spots and all vegetable or other perishable matter shall be entirely removed and the space refilled with suitable material.

When the concrete pavement is to be constructed over an old roadbed composed of gravel or macadam, the old roadbed shall be entirely loosened and the material spread for the full width of the roadbed and rolled. All interstices shall be filled with fine material and rolled to make a dense, tight surface of the roadbed.

17. *Acceptance.*—No concrete shall be deposited until the subgrade is checked and accepted by the engineer.

V. FORMS.

18. *Materials.*—Metal or wooden forms shall be free from warp, of sufficient strength to resist springing out of shape, and shall be equal in width to the thickness of the pavement at the edges. Wooden forms shall be of not less than two (2) inch stock, and shall be capped with two (2) inch angle iron.

19. *Setting.*—The forms shall be well staked or otherwise held to the established line and grades, and the upper edges shall conform to the established grade of the road.

20. *Treatment.*—All mortar and dirt shall be removed from forms before they are used.

VI. PAVEMENT SECTION.

21. *Width, Thickness of Concrete and Crown.*—The concrete pavement shall be .... feet wide, ... (........) inches in depth at center, and .... (........) inches in depth at the sides. The finished surface shall conform to the arc of a circle, as shown on the plans attached hereto.

> NOTE.—The thickness of the concrete at the edges shall be not less than six (6) inches. The crown shall be not less than one one-hundredth (1/100) nor more than one-fiftieth (1/50) of the width.

VII. JOINTS.

22. *Width and Location.*—Transverse joints shall be one-quarter (¼) inch in width and shall be placed across the pavement perpendicular to the center line, not more than thirty-six (36) feet apart. All joints shall extend through the entire thickness of the pavement and shall be perpendicular to its surface.

All catch basins, manhole tops, poles or other fixed objects which project through the pavement shall be separated from the concrete by joint filler.

23. *Joint Filler.*—All joints shall be formed by inserting during construction and leaving in place the required thickness of joint filler which shall extend through the entire thickness of the pavement.

24. *Protected Joints.**—The concrete at all transverse joints shall be protected with joint protection plates which shall be rigidly anchored to the concrete. The upper edges of the plates shall be even with each other and the adjoining surface of the concrete. All steel plates varying more than one-quarter (¼) inch from the finished surface of the concrete, as shown on the plans attached hereto, shall be ground to meet the specified requirements, or slabs in which such plates occur shall be removed and replaced with new material by the contractor at his expense.

25. *Unprotected Joints.**—All transverse joints shall extend through the entire thickness of the pavement and the filler shall project not less than one-half (½) inch above the finished surface. Before the road is opened to traffic, joint filler shall be cut off to a height of one-quarter (¼) inch above the surface of the road.

VIII. MEASURING MATERIALS AND MIXING CONCRETE.

26. *Measuring Materials.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 lbs. net) shall be considered one (1) cubic foot.

27. *Mixing.*—The materials shall be mixed in a batch mixer approved by the engineer, and irrespective of the size of the batch and rate of speed used, mixing shall continue after all materials are in the drum for at least one (1) minute before any part of the batch is discharged from the drum.

* When the specification "Protected Joints" is to be used, "Unprotected Joints" should be omitted, and *vice versa*.

The drum shall be completely emptied before receiving material for the succeeding batch. The volume of the mixed material used per batch shall not exceed the manufacturer's rated capacity of the drum in cubic feet of mixed material.

28. *Retempering.*—Retempering of mortar or concrete which has partly hardened; that is, remixing with or without additional materials or water, shall not be permitted.

29. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of portland cement to not more than two (2) cubic feet of fine aggregate and not more than three (3) cubic feet of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the coarse aggregate.

A cubic yard of concrete in place shall contain not less than one and seven-tenths (1.7) barrels of cement.

The engineer shall compare the calculated amount of cement required according to these specifications and plans attached hereto with the amounts actually used in each section of concrete between successive transverse joints, as determined by actual count of the number of sacks of cement used in each section. If the amount of cement used in any three (3) adjacent sections (between transverse joints) is less by more than two (2) per cent, or if the amount of cement used in any one section is less by more than five (5) per cent of the amount hereinbefore required, the contractor shall remove all such sections and replace the same with new materials, according to these specifications, at his expense.

30. *Consistency.*—The materials shall be mixed with only sufficient water to produce a concrete which will hold its shape when struck off with a template. The consistency shall not be such as to cause a separation of the mortar from the coarse aggregate in handling.

IX. REINFORCING.

31. *Reinforcing.*—Concrete pavements twenty (20) feet or more in width shall be reinforced. The reinforcement shall have a weight of not less than twenty-eight (28) pounds per one hundred (100) square feet. The ratio of effective areas of reinforcing members at right angles to each other may vary from 1 : 1 to 4 : 1. The spacing between parallel lines of reinforcing members shall not be more than eight (8) inches. A reduction of three (3) pounds from the weight specified shall be allowed for those types of reinforcement not requiring extra metal at intersections.

> NOTE.—The committee is of the opinion that the weight of reinforcement for streets over twenty-five (25) feet wide should be greater than twenty-eight (28) pounds per one hundred (100) square feet.

Reinforcing metal shall be placed not less than two (2) inches from the finished surface of the pavement and otherwise shall be placed as shown on the drawings attached hereto. The reinforcing metal shall extend to within two (2) inches of all joints, but shall not cross them. Adjacent widths of fabric shall be lapped not less than four (4) inches when the lap is made per-

pendicular to the center line of the pavement and not less than one (1) foot when the lap is made parallel to the center line of the pavement, and in most cases the use of reinforcement in pavements sixteen (16) feet wide or over is good practice.

X. PLACING CONCRETE.

32. *Placing Concrete.*—Immediately prior to placing the concrete, the subgrade shall be brought to an even surface. The surface of the subgrade shall be thoroughly wet but shall show no pools of water when the concrete is placed.

After mixing, the concrete shall be deposited rapidly upon the subgrade, to the required depth and for the entire width of the pavement in successive batches and in a continuous operation without the use of intermediate forms or bulkheads between expansion joints. If concrete is placed in two courses, as when reinforcement is used, any dirt, sand or dust which collects on the base course shall be removed before the top course is placed. The concrete above the reinforcement shall be placed immediately after mixing and in no case shall more than forty-five (45) minutes elapse between the time that the concrete below the reinforcement has been mixed and the concrete above the reinforcement is placed.

In case of a breakdown concrete shall be mixed by hand to complete the section or an intermediate transverse joint placed as hereinbefore specified at the point of stopping work. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used.

33. *Finishing.*—The concrete shall be brought to a proper contour by any means which will insure a compact dense surface. Any holes left by removing any material or device used in constructing joints shall be filled immediately with concrete from the latest batch deposited.

Concrete adjoining metal protection plates at transverse joints shall be dense in character and shall be given a smooth finish with a steel trowel for a distance of six (6) inches on each side of the joints. The concrete adjacent to unprotected joints shall be finished with a wood float, which is divided through the center and which will permit finishing on both sides of the filler at the same time.

Concrete shall be finished in a manner thoroughly to compact it and produce a surface free from depressions or inequalities of any kind. The finished surface of the pavement shall not vary more than one-quarter ($\frac{1}{4}$) inch from the specified contour.

> NOTE.—It is recommended that the contractor be required at the end of each day's work to stamp in the surface of the concrete with letters $1\frac{1}{2}$ to 2 in. high and $\frac{1}{4}$ in. deep, the date and his name.

XI. PROTECTION.

34. *Curing and Protection.*—Except as hereinafter specified, the surface of the pavement shall be sprayed with water as soon as the concrete is sufficiently hardened to prevent pitting, and shall be kept wet until an earth

or other approved covering is placed. As soon as it can be done without damaging the concrete, the surface of the pavement shall be covered with not less than two (2) inches of earth or other material approved by the engineer, which cover shall be kept wet for at least ten (10) days. When deemed necessary or advisable by the engineer, freshly-laid concrete shall be protected by canvas until such covering can be placed.

Under the most favorable conditions for hardening in hot weather the pavement shall be closed to traffic for at least fourteen (14) days, and in cool weather for an additional time, to be determined by the engineer.

When the average temperature is below fifty (50) degrees Fahrenheit, sprinkling and covering of the pavement may be omitted at the discretion of the engineer.

The contractor shall erect and maintain suitable barriers to protect the concrete from traffic and any part of the pavement damaged from traffic or other causes, occurring prior to its official acceptance, shall be repaired or replaced by the contractor at his expense, in a manner satisfactory to the engineer. Before the pavement is thrown open to traffic the covering shall be removed and disposed of as directed by the engineer.

35. *Cold Weather Work.*—Concrete shall not be mixed nor deposited when the temperature is below freezing.

If, at any time during the progress of the work, the temperature is, or in the opinion of the engineer will, within twenty-four (24) hours, drop to 35 degrees Fahrenheit, the water and aggregates shall be heated, and precautions taken to protect the work from freezing for at least ten (10) days. In no case shall concrete be deposited upon a frozen subgrade.

XII. SHOULDERS.

36. *Construction.*—When shoulders are required, they shall be built upon the properly prepared subgrade, as shown on the profile and cross-sectional drawings attached hereto. The work shall be done to the entire satisfaction of the engineer.

AMERICAN CONCRETE INSTITUTE.

STANDARD NO. 17.

ADOPTED BY LETTER BALLOT, APRIL 10, 1917.

STANDARD SPECIFICATIONS FOR ONE-COURSE CONCRETE STREET PAVEMENT.

I. MATERIALS.

1. *Cement.*—The cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement, adopted by the American Society for Testing Materials, September 1, 1916, with all subsequent amendments and additions thereto adopted by said Society and by this Institute (Standard No. 1).

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the engineer a fifty (50) pound sample of each of the fine and coarse aggregates proposed for use. These samples shall be tested and if found to pass the requirements of the specifications similar material shall be considered as acceptable for the work. Aggregates containing frost or lumps of frozen material shall not be used.

Fine Aggregate: Fine aggregate shall consist of natural sand or screenings from hard, tough, durable crushed rock or gravel, consisting of quartzite grains or other equally hard material graded from fine to coarse with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes per linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch, and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other deleterious matter nor more than three (3) per cent by weight of clay or loam. Routine field tests shall be made on fine aggregate as delivered. If there is more than seven (7) per cent of clay or loam by volume in one hour's settlement after shaking in an excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement, and three (3) parts fine aggregate, by weight, when made into briquettes, shall show a tensile strength (at seven (7) and twenty-eight (28) days) equal to or greater than the strength of briquettes composed of one (1) part of the same cement and three (3) parts standard Ottawa sand by weight. The percentage of water used in making the briquettes of cement and fine aggregate shall be such as to produce a mortar of the same consistency

as that of the Ottawa sand briquettes of standard consistency. In other respects all briquettes shall be made in accordance with the methods outlined in the Standard Specifications and Tests for Portland Cement adopted by the American Society for Testing Materials, Sept. 1, 1916.

Coarse Aggregate: Coarse aggregate shall consist of clean, hard, tough, durable crushed rock or pebbles graded in size, free from vegetable or other deleterious matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall be such as to pass a two (2) inch round opening and shall range from two (2) inches down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

3. *Mixed Aggregate.*—Crusher run stone, bank run gravel or artificially prepared mixtures of fine and coarse aggregate shall not be used.

4. *Water.*—Water shall be clean, free from oil, acid, alkali, or vegetable matter.

5. *Reinforcement.*—All reinforcement shall be free from excessive rust, scale, paint or coatings of any character which will tend to destroy the bond.

6. *Joint Filler.*—The filler for all transverse joints shall consist of prepared strips of fiber matrix and bitumen, or similar material of approved quality one-quarter ($\frac{1}{4}$) inch in thickness. Where the joints are protected with metal plates the joint filler shall be made to conform to the cross-section of the pavement, and where unprotected transverse joints are used, the width of the joint filler shall be at least one-half ($\frac{1}{2}$) inch greater than the thickness of the pavement at any point. The filler for longitudinal joints along the curb, where a separate curb is used, shall, at the discretion of the engineer, consist of the same material as specified for the transverse joints or of bitumen which will not become soft enough to flow in hot weather or brittle in cold weather. The thickness of longitudinal joints filled with bitumen shall be not less than one-quarter ($\frac{1}{4}$) inch nor more than three-quarters ($\frac{3}{4}$) inch. Prior to submitting bid the contractor shall obtain approval of the engineer for the joint filler which he proposes to use.

7. *Joint Protection Plates.*—Soft steel plates for the protection of the edges of the concrete at transverse joints shall be not less than two and one-half ($2\frac{1}{2}$) inches in depth and not less than one-eighth ($\frac{1}{8}$) nor more than one-quarter ($\frac{1}{4}$) inch average thickness. The plates shall be of such form as to provide for rigid anchorage to the concrete. The type and method of installation of joint protection plates shall be approved by the engineer.

II. GRADING.

8. *Defined.*—"Grading" shall include all cuts, fills, approaches and all earth or rock moving for whatever purpose where such work is an essential part of or necessary to the prosecution of the contract.

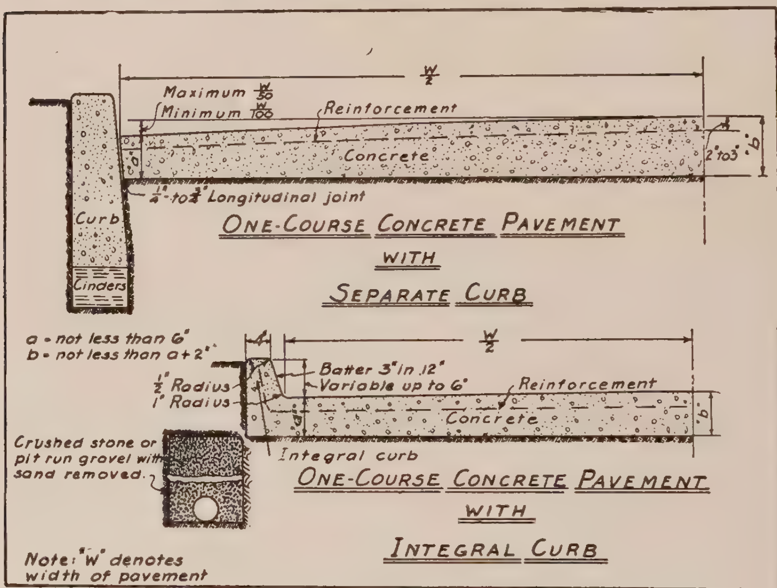
9. *Engineer's Stakes.*—Stakes will be set by the engineer for the center line, finished grades and other necessary points.

10. *Free Haul.*—Excess material shall be disposed of as directed by the engineer, the free haul not to exceed .... feet.

11. *Over-Haul.*—Materials hauled a greater distance than the free haul from the place of excavation shall be paid for at the rate of .... cents per cubic yard for each additional .... feet.

12. *Cuts and Fills.*—In cuts the final grade shall be obtained by rolling with a roller weighing not less than five (5) nor more than ten (10) tons. When a fill of one (1) foot or less is required to bring the surface to grade, all vegetable and other perishable matter shall be removed before making the fill.

Embankments shall be formed of earth or other approved materials and shall be constructed in successive layers, each of which shall extend entirely across the width to be filled. Each layer, which shall not exceed one (1) foot in depth, shall be thoroughly rolled with a roller weighing not less than five



(5) nor more than ten (10) tons before the succeeding layer is placed. The roller shall pass over the entire area of each layer of the fill at least twice. The use of muck, quicksand, soft clay, spongy or perishable material, which will not consolidate under the roller, is prohibited.

When the material excavated from the cuts is not sufficient to make the fills shown on the plans, the contractor shall furnish the necessary extra material to bring the fills to the proper width and grade. When the earth work is completed the cross-section of the roadbed shall conform to the cross-sectional drawings and profile attached hereto.

All approaches connecting the specified pavement with other streets or alleys intersecting shall also be cut or filled, and secured from settlement, to form a slope of not more than one (1) vertical to ten (10) horizontal, as shown on the profile and plans attached hereto.

III. DRAINAGE.

13. *Drainage.*—The contractor shall construct tile or other drains as shown in the drawings attached hereto. Tile to be laid in a trench at least .... (........) inches wide, and .... (........) feet deep below the top of the adjacent curb. Such trench shall be back-filled with crushed stone or pit run gravel with sand removed, which after light tamping shall be .... (........) inches in depth.

14. *Catch Basins.*—All catch basin and manhole tops and all covers or openings of any kind shall be adjusted to the grade by the contractor at the price shown under this item in his bid.

IV. SUBGRADE.

15. *Construction.*—The subgrade shall be brought to a firm density by rolling the entire area with a self-propelled roller. All portions of the surface of the subgrade which are inaccessible to the roller shall be thoroughly tamped with a hand tamp weighing not less than fifty (50) pounds, the face of which shall not exceed one hundred (100) square inches in area. All soft, spongy or yielding spots and all vegetable or other perishable matter shall be entirely removed and the space refilled with suitable material.

When the concrete pavement is to be constructed over an old roadbed composed of gravel or maçadam, the old roadbed shall be entirely loosened and the material spread for the full width of the roadbed and rolled. All interstices shall be filled with fine material and rolled to make a dense, tight surface of the roadbed.

16. *Acceptance.*—No concrete shall be deposited until the subgrade is checked and accepted by the engineer.

V. FORMS.

17. *Materials.*—Where forms are required, they shall be free from warp and of sufficient strength to resist springing out of shape. Wooden forms shall be of not less than two (2) inch stock.

18. *Setting.*—The forms shall be well staked or otherwise held to the established line and grades. Where the curb is to be constructed integrally with the pavement, the upper edge of the side forms shall conform to the top of the curb.

19. *Treatment.*—All mortar and dirt shall be removed from forms before they are used.

VI. PAVEMENT SECTION.

20. *Width, Thickness of Concrete and Crown.*—The concrete pavement shall be .... (........) feet wide from face to face of curb,.... .... (........) inches in depth at the center and .... (........) inches in depth at the sides. The finished surface shall conform to the lines as shown on the plans attached hereto.

NOTE.—The thickness of the concrete at the sides shall be not less than six (6) inches and at the center not less than two (2) inches more

than the thickness at the sides. When pavements twenty (20) feet or less in width are to be built on approximately level ground and a flat subgrade is to be used, sufficient fall for drainage at the sides of the pavement along the curb shall be provided by giving the roadbed the same grade as that proposed for the gutter. The crown shall be not less than one one-hundredth (1/100) nor more than one-fiftieth (1/50) of the width.

VII. JOINTS.

21. *Width and Location.*—Transverse joints shall be one-quarter ($\frac{1}{4}$) inch in width and shall be placed across the pavement perpendicular to the center line, not more than thirty-six (36) feet apart. A longitudinal joint not less than one-quarter ($\frac{1}{4}$) inch wide shall be constructed between the curb and the pavement where a separate curb is used. All joints shall extend through the entire thickness of the pavement and curb (when integral curb is specified) and shall be perpendicular to the surface of the pavement. In pavements with integral curb the joint shall be continuous in a straight line through pavement and curb.

All catch basins, manhole tops, poles or other fixed objects which project through the pavement shall be separated from the concrete by joint filler.

22. *Joint Filler.*—All transverse joints shall be formed by inserting during construction and leaving in place the required thickness of prepared strips of fiber matrix and bitumen or similar material of approved quality which shall extend through the entire thickness of the pavement and the entire thickness and height of the integral curb when the latter is specified.

Longitudinal joints along the curb, where a separate curb is specified, shall, at the discretion of the engineer, be formed in the same manner as transverse joints or constructed by filling with bitumen as before specified.

23. *Protected Joints.**—The concrete at all transverse joints shall be protected with joint protection plates which shall be rigidly anchored to the concrete. The upper edges of the plates shall be even with each other and the adjoining surface of the concrete. All steel plates varying more than one-quarter ($\frac{1}{4}$) inch from the finished surface of the concrete, as shown on the plans attached hereto, shall be ground to meet the specified requirements, or slabs in which such plates occur shall be removed and replaced with new material by the contractor at his expense.

24. *Unprotected Joints.**—All transverse joints shall extend through the entire thickness of the pavement and the filler shall project not less than one-half ($\frac{1}{2}$) inch above the finished surface. Before the pavement is opened to traffic joint filler shall be cut off to a height of one-quarter ($\frac{1}{4}$) inch above the surface of the pavement.

VIII. MEASURING MATERIALS AND MIXING CONCRETE.

25. *Measuring Materials.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate and

* When the specification "Protected Joints" is to be used, "Unprotected Joints" should be omitted, and *vice versa*.

uniform proportions of each of the materials at all times. A sack of portland cement (94 pounds net) shall be considered one (1) cubic foot.

26. *Mixing.*—The materials shall be mixed in a batch mixer approved by the engineer, and irrespective of the size of the batch and rate of speed used, mixing shall continue after all materials are in the drum for at least one (1) minute before any part of the batch is discharged from the drum. The drum shall be completely emptied before receiving material for the succeeding batch. The volume of the mixed material used per batch shall not exceed the manufacturer's rated capacity of the drum in cubic feet of mixed material.

27. *Retempering.*—Retempering of mortar or concrete which has partly hardened, that is, remixing with or without additional materials or water shall not be permitted.

28. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of Portland cement to not more than two (2) cubic feet of fine aggregate and not more than three (3) cubic feet of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) of the volume of the coarse aggregate.

A cubic yard of concrete in place, including that in integral curbs when same are used, shall contain not less than one and seven-tenths (1.7) barrels of cement.

The engineer shall compare the calculated amount of cement required according to these specifications and plans attached hereto with the amounts actually used in each section of concrete (including integral curbs when used) between successive transverse joints, as determined by actual count of the number of sacks of cement used in each section. If the amount of cement used in any three (3) adjacent sections (between transverse joints) is less by more than two (2) per cent, or if the amount of cement used in any one section is less by more than five (5) per cent of the amount hereinbefore required, the contractor shall remove all such sections and replace the same with new materials, according to these specifications, at his expense.

29. *Consistency.*—The materials for the pavement shall be mixed with only sufficient water to produce a concrete which will hold its shape when struck off with a template. The consistency shall not be such as to cause a separation of the mortar from the coarse aggregate in handling.

IX. REINFORCING.

30. *Reinforcing.*—Concrete pavements twenty (20) feet or more in width shall be reinforced. The reinforcement shall have a weight of not less than twenty-eight (28) pounds per one hundred (100) square feet. The ratio of effective areas of reinforcing members at right angles to each other may vary from 1 : 1 to 4 : 1. The spacing between parallel lines of reinforcing members shall not be more than eight (8) inches. A reduction of three (3) pounds from the weight specified shall be allowed for those types of reinforcement not requiring extra metal at intersections.

NOTE.—The committee is of the opinion that the weight of reinforcement for streets over twenty-five (25) feet wide should be greater than twenty-eight (28) pounds per one hundred (100) square feet.

Reinforcing metal shall not be placed less than two (2) inches from the finished surface of the pavement and otherwise shall be placed as shown on the drawings attached hereto. The reinforcing metal shall extend to within two (2) inches of all joints, but shall not cross them. Adjacent widths of fabric shall be lapped not less than four (4) inches when the lap is made perpendicular to the center line of the pavement and not less than one (1) foot when the lap is made parallel to the center line of the pavement and in most cases the use of reinforcement in pavements sixteen (16) feet wide or over is good practice.

X. PLACING CONCRETE.

31. *Placing Concrete.*—Immediately prior to placing the concrete, the subgrade shall be brought to an even surface. The surface of the subgrade shall be thoroughly wet but shall show no pools of water when the concrete is placed.

After mixing, the concrete shall be deposited rapidly upon the subgrade, to the required depth and for the entire width of the pavement in successive batches and in a continuous operation without the use of intermediate forms or bulkheads between expansion joints. If the concrete is placed in two courses, as when reinforcement is used, any dirt, sand or dust which collects on the base course shall be removed before the top course is placed. The concrete above the reinforcement shall be placed immediately after mixing and in no case shall more than forty-five (45) minutes elapse between the time that the concrete below the reinforcement has been mixed and the concrete above the reinforcement is placed.

In case of a breakdown, concrete shall be mixed by hand to complete the section or an intermediate transverse joint placed as hereinbefore specified at the point of stopping work. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used in the work.

32. *Finishing.*—The concrete shall be brought to a proper contour by any means which will insure a compact dense surface. Any holes left by removing any material or device used in constructing joints shall be filled immediately with concrete from the latest batch deposited. Concrete adjoining metal protection plates at transverse joints shall be dense in character and shall be given a smooth finish with a steel trowel for a distance of six (6) inches on each side of the joints. The concrete adjacent to unprotected joints shall be finished with a wood float, which is divided through the center and which will permit finishing on both sides of the filler at the same time. The concrete for a distance of eighteen (18) inches out from the curb line shall be finished with a steel trowel.

Concrete shall be finished in a manner thoroughly to compact it and produce a surface free from depressions or inequalities of any kind. The concrete shall be finished with a wood float in a manner to thoroughly compact it and produce a surface free from depressions or inequalities of any kind. The finished surface of the pavement shall not vary more than one-quarter ($\frac{1}{4}$) inch from the specified contour.

> NOTE.—It is recommended that the contractor be required at the end of each day's work to stamp in the surface of the concrete with letters $1\frac{1}{2}$ to 2 inches high and $\frac{1}{4}$ inch deep, the date and his name.

XI. INTEGRAL CURB.*

33. *Construction.*—An integral curb shall be constructed, as shown on the attached drawings, to the established grade and in a continuous line on each side of the street .... (........) feet from and parallel with the center line thereof, except at all intersections of streets, alleys, and driveways where it shall be returned to the street line, and at these intersections it shall be rounded to such a radius as the engineer may direct.

The concrete for integral curbs shall be of the same materials and proportions as that required for the pavement.

After striking off the pavement, the concrete for that portion of the integral curbs above the gutter line shall be immediately deposited.

The top and inside surface of integral curb shall be given a smooth finish and completed with the pavement to the point of stopping each day's work.

XII. PROTECTION.

34. *Curing and Protection.*—Except as hereinafter specified, the surface of the pavement shall be sprayed with water as soon as the concrete is sufficiently hardened to prevent pitting, and shall be kept wet until an earth or other approved covering is placed. As soon as it can be done without damaging the concrete, the surface of the pavement shall be covered with not less than two (2) inches of earth or other material approved by the engineer, which cover shall be kept wet for at least ten (10) days. When deemed necessary or advisable by the engineer, freshly laid concrete shall be protected by canvas until such covering can be placed.

Under the most favorable conditions for hardening in hot weather, the pavement shall be closed to traffic for at least fourteen (14) days, and in cool weather for an additional time, to be determined by the engineer.

When the average temperature is below fifty (50) degrees Fahrenheit, sprinkling and covering of the pavement may be omitted at the discretion of the engineer.

The contractor shall erect and maintain suitable barriers to protect the concrete from traffic and any part of the pavement damaged from traffic or other causes, occurring prior to its official acceptance, shall be repaired or replaced by the contractor at his expense, in a manner satisfactory to the engineer. Before the pavement is thrown open to traffic the covering shall be removed and disposed of as directed by the engineer.

35. *Cold Weather Work.*—Concrete shall not be mixed nor deposited when the temperature is below freezing.

If, at any time during the progress of the work, the temperature is, or in the opinion of the engineer will within twenty-four (24) hours drop to 35 degrees Fahrenheit, the water and aggregates shall be heated, and precautions taken to protect the work from freezing for at least ten (10) days. In no case shall concrete be deposited upon a frozen subgrade.

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* Article 33 to be omitted if separate curb is specified.

AMERICAN CONCRETE INSTITUTE.

STANDARD NO. 18.

ADOPTED BY LETTER BALLOT, APRIL 10, 1917.

STANDARD SPECIFICATIONS FOR TWO-COURSE CONCRETE STREET PAVEMENT.

I. Materials.

1. *Cement.*—The cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement, adopted by the American Society for Testing Materials, September 1, 1916, with all subsequent amendments and additions thereto adopted by said Society and by this Institute (Standard No. 1).

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the engineer a fifty (50) pound sample of each kind of aggregate proposed for use. These samples shall be tested and if found to pass the requirements of the specifications similar material shall be considered as acceptable for the work. Aggregates containing frost or lumps of frozen material shall not be used.

Fine Aggregate: Fine aggregate shall consist of natural sand or screenings from hard, tough, durable crushed rock or gravel, consisting of quartzite grains or other equally hard material graded from fine to coarse with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes per linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch, and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other deleterious matter, nor more than three (3) per cent by weight of clay or loam. Routine field tests shall be made on fine aggregate as delivered. If there is more than seven (7) per cent of clay or loam by volume in one (1) hour's settlement after shaking in an excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement, and three (3) parts fine aggregate, by weight, when made into briquettes, shall show a tensile strength (at seven (7) and twenty-eight (28) days) equal to or greater than the strength of briquettes composed of one (1) part of the same cement and three (3) parts standard Ottawa sand by weight. The percentage of water used in making the briquettes of cement

and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquettes of standard consistency. In other respects all briquettes shall be made in accordance with the methods outlined in the Standard Specifications and Tests for Portland Cement adopted by the American Society for Testing Materials, Sept. 1, 1916.

Coarse Aggregate: Coarse aggregate shall consist of clean, durable crushed rock or pebbles graded in size, free from vegetable or other deleterious matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall be such as to pass a two (2) inch round opening and shall range from two (2) inches down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch.

No. 1 Aggregate for Wearing Course: No. 1 aggregate for the wearing course shall consist of clean, hard, tough, durable crushed rock or pebbles, free from vegetable matter, and shall contain no soft, flat or elongated particles. It shall pass when dry a screen having one-half ($\frac{1}{2}$) inch openings and not more than ten (10) per cent shall pass a screen having one-quarter ($\frac{1}{4}$) inch openings.

No. 2 Aggregate for Wearing Course: No. 2 aggregate for the wearing course shall consist of clean, hard, tough, durable crushed rock or pebbles graded in size, free from vegetable or other deleterious matter, and shall contain no soft, flat or elongated particles. The size of No. 2 coarse aggregate shall be such as to pass a one (1) inch round opening and shall range from one (1) inch down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch and no intermediate sizes shall be removed.

3. *Mixed Aggregate.*—Crusher run stone, bank run gravel or artificially prepared mixtures of fine and coarse aggregate shall not be used

4. *Water.*—Water shall be clean, free from oil, acid, alkali or vegetable matter.

5. *Reinforcement.*—All reinforcement shall be free from excessive rust, scale, paint or coatings of any character which will tend to destroy the bond.

6. *Joint Filler.*—The filler for all transverse joints shall consist of prepared strips of fiber matrix and bitumen or similar material of approved quality, one-quarter ($\frac{1}{4}$) inch in thickness. Where the joints are protected with metal plates the joint filler shall be made to conform to the cross-section of the pavement and where unprotected transverse joints are used the width of the joint filler shall be at least one-half ($\frac{1}{2}$) inch greater than the thickness of the pavement at any point. The filler for longitudinal joints along the curb, where a separate curb is used, shall, at the discretion of the engineer, consist of the same material as specified for the transverse joints, or of bitumen which will not become soft enough to flow in hot weather or brittle in cold weather. The thickness of longitudinal joints filled with bitumen shall be not less than one-quarter ($\frac{1}{4}$) inch nor more than three-quarters ($\frac{3}{4}$) inch. Prior to submitting bid the contractor shall obtain approval of the engineer for the joint filler which he proposes to use.

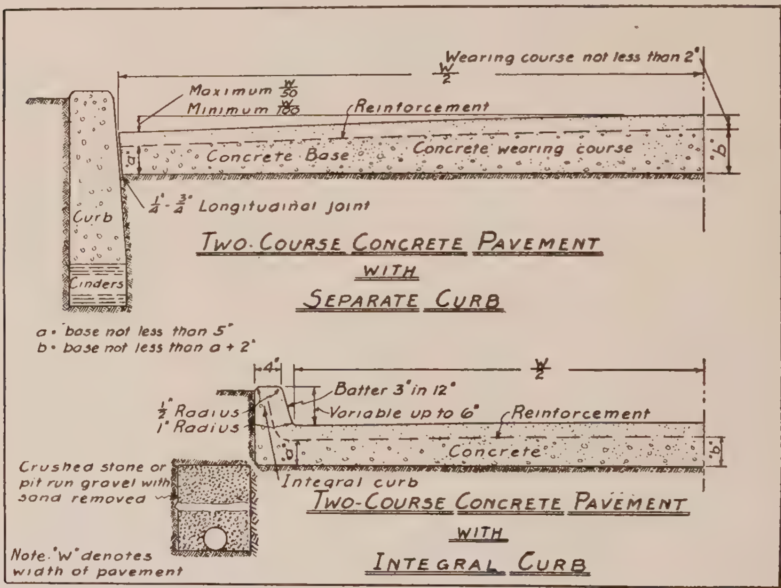
7. *Joint Protection Plates.*—Soft steel plates for the protection of the edges of the concrete at transverse joints shall be not less than two and one-half ($2\frac{1}{2}$) inches in depth and not less than one-eighth ($\frac{1}{8}$) inch nor more than

one-quarter ($\frac{1}{4}$) inch average thickness. The plates shall be of such form as to provide for rigid anchorage to the concrete. The type and method of installation of joint protection plates shall be approved by the engineer.

II. GRADING.

8. *Defined.*—"Grading" shall include all cuts, fills, approaches and all earth or rock moving for whatever purpose where such work is an essential part of or necessary to the prosecution of the contract.

9. *Engineer's Stakes.*—Stakes will be set by the engineer for the center line, finished grades and other necessary points.



10. *Free Haul.*—Excess material shall be disposed of as directed by the engineer, the free haul not to exceed .... feet.

11. *Over-Haul.*—Materials hauled a greater distance than the free haul from the place of excavation shall be paid for at the rate of .... cents per cubic yard for each additional .... feet.

12. *Cuts and Fills.*—In cuts the final grade shall be obtained by rolling with a roller weighing not less than five (5) nor more than ten (10) tons. When a fill of one (1) foot or less is required to bring the surface to grade, all vegetable or other perishable matter shall be removed before making the fill.

Embankments shall be formed of earth or other approved materials and shall be constructed in successive layers, each of which shall extend entirely across the width to be filled. Each layer, which shall not exceed one (1)

foot in depth, shall be thoroughly rolled with a roller weighing not less than five (5) nor more than ten (10) tons before the succeeding layer is placed. The roller shall pass over the entire area of each layer of the fill at least twice. The use of muck, quicksand, soft clay, spongy or perishable material, which will not consolidate under the roller, is prohibited.

When the material excavated from the cuts is not sufficient to make the fills shown on the plans, the contractor shall furnish the necessary extra material to bring the fills to the proper width and grade. When the earthwork is completed the cross-section of the roadbed shall conform to the cross-sectional drawings and profile attached hereto.

All approaches connecting the specified pavement with other streets or alleys intersecting shall also be cut or filled and secured from settlement to form a slope of not more than one (1) vertical to ten (10) horizontal, as shown on the profile and plans attached hereto.

III. DRAINAGE.

13. *Drainage.*—The contractor shall construct tile or other drains as shown in the drawings attached hereto. Tile to be laid in a trench at least .... (........) inches wide, and .... (........) feet deep below the top of the adjacent curb. Such trench shall be backfilled with crushed stone or pit run gravel with sand removed, which after light tamping shall be .... (........) inches in depth.

14. *Catch Basins.*—All catch basin and manhole tops and all covers of openings of any kind shall be adjusted to the grade by the contractor at the price shown under this item in his bid.

IV. SUBGRADE.

15. *Construction.*—The subgrade shall be brought to a firm density by rolling the entire area with a self-propelled roller. All portions of the surface of the subgrade which are inaccessible to the roller shall be thoroughly tamped with a hand tamp weighing not less than fifty (50) pounds, the face of which shall not exceed one hundred (100) square inches in area. All soft, spongy or yielding spots and all vegetable or other perishable matter shall be entirely removed and the space refilled with suitable material.

When the concrete pavement is to be constructed over an old roadbed composed of gravel or macadam, the old roadbed shall be entirely loosened and the material spread for the full width of the roadbed and rolled. All interstices shall be filled with fine material and rolled to make a dense, tight surface of the roadbed.

16. *Acceptance.*—No concrete shall be deposited until the subgrade is checked and accepted by the engineer.

V. FORMS.

17. *Materials.*—Where forms are required, they shall be free from warp and of sufficient strength to resist springing out of shape. Wooden forms shall be of not less than two (2) inch stock.

18. *Setting.*—The forms shall be well staked or otherwise held to the established line and grades. Where the curb is to be constructed integrally with the pavement, the upper edge of the side forms shall conform to the top of the curb.

19. *Treatment.*—All mortar and dirt shall be removed from forms before they are used.

VI. PAVEMENT SECTION.

20. *Width, Thickness of Concrete and Crown.*—The concrete pavement shall be .... (........) feet wide from face to face of curb. The base of the concrete pavement shall be .... (........) inches in depth at the center and .... (........) inches in depth at the sides. The wearing course shall be .... (........) inches uniform thickness. The finished surface shall conform to the lines as shown on the plans attached hereto.

> NOTE.—The thickness of the concrete base at the sides shall be not less than five (5) inches and at the center not less than two (2) inches more than the thickness at the sides. The thickness of the wearing course shall be not less than two (2) inches. When pavements twenty (20) feet or less in width are to be built on approximately level ground and a flat subgrade is to be used, sufficient fall for drainage at the sides of the pavement along the curb shall be provided by giving the roadbed the same grade as that proposed for the gutter. The crown shall be not less than one one-hundredth (1/100) nor more than one-fiftieth (1/50) of the width.

VII. JOINTS.

21. *Width and Location.*—Transverse joints shall be one-quarter ($\frac{1}{4}$) inch in width and shall be placed across the pavement perpendicular to the center line, not more than thirty-six (36) feet part. A longitudinal joint not less than one-quarter ($\frac{1}{4}$) inch wide shall be constructed between the curb and the pavement where a separate curb is used. All joints shall extend through the entire thickness of the pavement and curb (when integral curb is specified) and shall be perpendicular to the surface of the pavement. In pavements with integral curb the joint shall be continuous in a straight line through pavement and curb.

All catch basins, manhole tops, poles or other fixed objects which project through the pavement shall be separated from the concrete by joint filler.

22. *Joint Filler.*—All transverse joints shall be formed by inserting during construction and leaving in place the required thickness of prepared strips of fiber matrix and bitumen, or similar material of approved quality, which shall extend through the entire thickness of the pavement, and the entire thickness and height of the integral curb when the latter is specified.

Longitudinal joints along the curb, where a separate curb is specified, shall, at the discretion of the engineer, be formed in the same manner as transverse joints or constructed by filling with bitumen as before specified.

23. *Protected Joints.**—The concrete at all transverse joints shall be protected with joint protection plates which shall be rigidly anchored to the concrete. The upper edges of the plates shall be even with each other and the adjoining surface of the concrete. All steel plates varying more than one-quarter ($\frac{1}{4}$) inch from the finished surface of the concrete, as shown on the plans attached hereto, shall be ground to meet the specified requirements, or slabs in which such plates occur shall be removed and replaced with new material by the contractor at his expense.

24. *Unprotected Joints.**—All transverse joints shall extend through the entire thickness of the pavement and the filler shall project not less than one-half ($\frac{1}{2}$) inch above the finished surface. Before the pavement is opened to traffic joint filler shall be cut off to a height of one-quarter ($\frac{1}{4}$) inch above the surface of the pavement.

VIII. MEASURING MATERIALS AND MIXING CONCRETE.

25. *Measuring Materials.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 pounds net) shall be considered one (1) cubic foot.

26. *Mixing.*—The materials shall be mixed in a batch mixer approved by the engineer, and irrespective of the size of the batch and rate of speed used, mixing shall continue after all materials are in the drum for at least one (1) minute before any part of the batch is discharged from the drum. The drum shall be completely emptied before receiving material for the succeeding batch. The volume of the mixed material used per batch shall not exceed the manufacturer's rated capacity of the drum in cubic feet of mixed material.

27. *Retempering.*—Retempering of mortar or concrete which has partly hardened, that is, remixing with or without additional materials or water, shall not be permitted.

28. *Consistency.*—The materials for the pavement shall be mixed with only sufficient water to produce a concrete which will hold its shape when struck off with a template. The consistency shall not be such as to cause a separation of the mortar from the aggregate in handling.

29. *Cement Required.*—A cubic yard of concrete base in place shall contain not less than one and four-tenths (1.4) barrels of cement. A cubic yard of "No. 1 Mix Wearing Course" in place shall contain not less than two and ninety-seven-hundredths (2.97) barrels of cement, and a cubic yard of "No. 2 Mix Wearing Course" in place shall contain not less than two and one-tenth (2.1) barrels of cement.

The engineer shall compare the calculated amount of cement required according to these specifications and plans attached hereto with the amounts actually used in each section of concrete (including integral curbs when used) between successive transverse joints, as determined by actual count of the

* When the specification "Protected Joints" is to be used, "Unprotected Joints" should be omitted, and *vice versa*.

number of sacks of cement used in each section. If the amount of cement used in any three (3) adjacent sections (between transverse joints) is less by more than two (2) per cent, or if the amount of cement used in any one section is less by more than five (5) per cent of the amount hereinbefore required, the contractor shall remove all such sections and replace the same with new materials, according to these specifications, at his expense.

IX. Reinforcing.

30. *Reinforcing.*—Concrete pavements twenty (20) feet or more in width shall be reinforced. The reinforcement shall have a weight of not less than twenty-eight (28) pounds per one hundred (100) square feet. The ratio of effective areas of reinforcing members at right angles to each other may vary from 1 : 1 to 4 : 1. The spacing between parallel lines of reinforcing members shall not be more than eight (8) inches. A reduction of three (3) pounds from the weight specified shall be allowed for those types of reinforcement not requiring extra metal at intersections.

> Note.—The committee is of the opinion that the weight of reinforcement for streets over twenty-five (25) feet wide should be greater than twenty-eight (28) pounds per one hundred (100) square feet.

Reinforcing metal shall be placed between base and wearing course and shall not be less than two (2) inches from the finished surface of the pavement and otherwise shall be placed as shown on the drawings. The reinforcing metal shall extend to within two (2) inches of all joints, but shall not cross them. Adjacent widths of fabric shall be lapped not less than four (4) inches when the lap is made perpendicular to the center line of the pavement and not less than one (1) foot when the lap is made parallel to the center line of the pavement and in most cases the use of reinforcement in pavements sixteen (16) feet wide or over is good practice.

X. Placing Concrete.

Concrete for Base.

31. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of portland cement to not more than two and a half ($2\frac{1}{2}$) cubic feet of fine aggregate, and not more than four (4) cubic feet of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the coarse aggregate.

32. *Placing Concrete.*—Immediately prior to placing the concrete, the subgrade shall be brought to an even surface. The surface of the subgrade shall be thoroughly wet but shall show no pools of water when the concrete is placed.

After mixing, the concrete shall be deposited rapidly upon the subgrade, to the required depth and for the entire width of the pavement in successive batches and in a continuous operation without the use of intermediate forms or bulkheads between expansion joints.

The concrete shall be brought to an even surface, the thickness of the

wearing course below the established grade of the pavement. If dirt, sand or dust collects on the base it shall be removed before the wearing course is applied. The reinforcing metal shall be placed upon and slightly pressed into the concrete base immediately after it is placed.

In case of a breakdown concrete shall be mixed by hand to complete the section, or an intermediate transverse joint placed, as hereinbefore specified, at the point of stopping work. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used.

Concrete for Wearing Course.

33. *Proportions for Mixture No. 1.**—The concrete for the wearing course shall be mixed in the manner hereinbefore specified in the proportions of one (1) sack of portland cement to not more than one (1) cubic foot of fine aggregate, and not more than one and one-half ($1\frac{1}{2}$) cubic feet of "No. 1 Aggregate for Wearing Course" and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the "No. 1 Aggregate for Wearing Course."

34. *Proportions for Mixture No. 2.**—The concrete for the wearing course shall be mixed in the manner hereinbefore specified in the proportions of one (1) sack of portland cement to not more than one and one-half ($1\frac{1}{2}$) cubic feet of fine aggregate and not more than two and one-half ($2\frac{1}{2}$) cubic feet of "No. 2 Aggregate for Wearing Course" hereinbefore specified, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of "No. 2 Aggregate for Wearing Course."

35. *Placing.*—The wearing course shall be placed immediately after mixing and in no case shall more than forty-five (45) minutes elapse between the time that the concrete for the base has been mixed and the time the wearing course is placed.

36. *Finishing.*—The wearing course shall be brought to a proper contour by any means which will insure a compact dense surface. Any holes left by removing any material or device used in constructing joints shall be filled immediately with concrete from the latest batch deposited. The concrete adjacent to unprotected joints shall be finished with a wood float, which is divided through the center and which will permit finishing on both sides of the filler at the same time. The concrete for a distance of eighteen (18) inches out from the curb line shall be finished with a steel trowel.

The wearing course shall be finished in a manner thoroughly to compact it and produce a surface free from depressions or inequalities of any kind. The finished surface of the pavement shall not vary more than one-quarter ($\frac{1}{4}$) inch from the specified contour.

NOTE.—It is recommended that the contractor be required at the end of each day's work to stamp in the surface of the concrete with letters $1\frac{1}{2}$ to 2 inches high and $\frac{1}{4}$ inch deep, the date and his name.

* When Article 33 is specified, Article 34 should be omitted, and *vice versa*.

XI. INTEGRAL CURB.*

37. *Construction.*—An integral curb shall be constructed, as shown on the attached drawings, to the established grade and in a continuous line on each side of the street .... (........) feet from and parallel with the center line thereof, except at all intersections of streets, alleys and driveways, where it shall be returned to the street line, and at these intersections it shall be rounded to such a radius as the engineer may direct.

The concrete for that portion of the integral curbs above the base shall be of the same materials and proportions as that specified for the wearing course.

After striking off the pavement, the concrete for that portion of the integral curbs above the gutter line shall be immediately deposited.

The top and inside surface of integral curbs shall be given a smooth finish and completed with the pavement to the point of stopping each day's work.

XII. PROTECTION.

38. *Curing and Protection.*—Except as hereinafter specified, the surface of the pavement shall be sprayed with water as soon as the concrete is sufficiently hardened to prevent pitting, and shall be kept wet until an earth or other approved covering is placed. As soon as it can be done without damaging the concrete, the surface of the pavement shall be covered with not less than two (2) inches of earth or other material approved by the engineer, which cover shall be kept wet for at least ten (10) days. When deemed necessary or advisable by the engineer, freshly laid concrete shall be protected by canvas until such covering can be placed.

Under the most favorable conditions for hardening in hot weather the pavement shall be closed to traffic for at least fourteen (14) days, and in cool weather for an additional time, to be determined by the engineer.

When the average temperature is below 50 degrees Fahrenheit, sprinkling and covering of the pavement may be omitted at the discretion of the engineer.

The contractor shall erect and maintain suitable barriers to protect the concrete from traffic and any part of the pavement damaged from traffic or other causes, occurring prior to its official acceptance, shall be repaired or replaced by the contractor at his expense, in a manner satisfactory to the engineer. Before the pavement is thrown open to traffic the covering shall be removed and disposed of as directed by the engineer.

39. *Cold Weather Work.*—Concrete shall not be mixed nor deposited when the temperature is below freezing.

If, at any time during the progress of the work, the temperature is, or in the opinion of the engineer will within twenty-four (24) hours drop to 35 degrees Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least ten (10) days. In no case shall concrete be deposited upon a frozen subgrade.

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* Article 37 to be omitted if separate curb is specified.

AMERICAN CONCRETE INSTITUTE.

STANDARD NO. 19.

ADOPTED BY LETTER BALLOT, APRIL 10, 1917.

STANDARD SPECIFICATIONS FOR ONE-COURSE CONCRETE ALLEY PAVEMENT.

I. Materials.

1. *Cement.*—The cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement, adopted by the American Society for Testing Materials, September 1, 1916, with all subsequent amendments and additions thereto adopted by said Society and by this Institute (Standard No. 1).

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the engineer a fifty (50) pound sample of each of the fine and coarse aggregates proposed for use. These samples shall be tested and if found to pass the requirements of the specifications similar material shall be considered as acceptable for the work. Aggregates containing frost or lumps of frozen material shall not be used.

Fine Aggregate: Fine aggregate shall consist of natural sand or screenings from hard, tough, durable crushed rock or gravel, consisting of quartzite grains or other equally hard material graded from fine to coarse with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes per linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch, and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other deleterious matter nor more than three (3) per cent by weight of clay or loam. Routine field tests shall be made on fine aggregate as delivered. If there is more than seven (7) per cent of clay or loam by volume in one (1) hour's settlement after shaking in an excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement and three (3) parts fine aggregate, by weight, when made into briquettes, shall show a tensile strength (at seven (7) and twenty-eight (28) days) equal to or greater than the strength of briquettes composed of one (1) part of the same cement and three (3) parts standard Ottawa sand, by weight. The percentage of water used in mixing the briquettes of

cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquettes of standard consistency. In other respects all briquettes shall be made in accordance with the methods outlined in the Standard Specifications and Tests for Portland Cement adopted by the American Society for Testing Materials, September 1, 1916.

Coarse Aggregate: Coarse aggregate shall consist of clean, hard, tough, durable crushed rock or pebbles graded in size, free from vegetable or other deleterious matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall be such as to pass a two (2) inch round opening and shall range from two (2) inches down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

3. *Mixed Aggregate.*—Crusher run stone, bank run gravel or artificially prepared mixtures of fine and coarse aggregate shall not be used.

4. *Water.*—Water shall be clean, free from oil, acid, alkali or vegetable matter.

5. *Reinforcement.*—All reinforcement shall be free from excessive rust, scale, paint or coatings of any character which will tend to destroy the bond.

6. *Joint Filler.*—The filler for all transverse joints shall consist of prepared strips of fiber matrix and bitumen or similar material of approved quality one-quarter ($\frac{1}{4}$) inch in thickness. Where the joints are protected with metal plates the joint filler shall be made to conform to the cross-section of the pavement and where unprotected transverse joints are used the width of the joint filler shall be at least one-half ($\frac{1}{2}$) inch greater than the thickness of the pavement at any point. The filler for longitudinal joints shall, at the discretion of the engineer, consist of the same material as specified for the transverse joints or of bitumen which will not become soft enough to flow in hot weather or brittle in cold weather. The thickness of longitudinal joints filled with bitumen shall be not less than one-quarter ($\frac{1}{4}$) inch nor more than three-quarters ($\frac{3}{4}$) inch. Prior to submitting bid the contractor shall obtain approval of the engineer for the joint filler which he proposes to use.

7. *Joint Protection Plates.*—Soft steel plates for the protection of the edges of the concrete at transverse joints shall be not less than two and one-half ($2\frac{1}{2}$) inches in depth and not less than one-eighth ($\frac{1}{8}$) nor more than one-quarter ($\frac{1}{4}$) inch average thickness. The plates shall be of such form as to provide for rigid anchorage to the concrete. The type and method of installation of joint protection plates shall be approved by the engineer.

II. GRADING.

8. *Defined.*—"Grading" shall include all cuts, fills, approaches and all earth or rock moving for whatever purpose where such work is an essential part of or necessary to the prosecution of the contract.

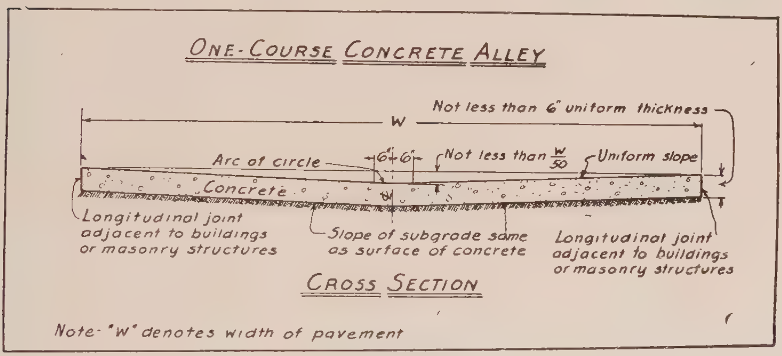
9. *Engineer's Stakes.*—Stakes will be set by the engineer for the center line, finished grades and other necessary points.

10. *Free Haul.*—Excess material shall be disposed of as directed by the engineer, the free haul not to exceed .... feet.

11. *Over-Haul.*—Materials hauled a greater distance than the free haul from the place of excavation shall be paid for at the rate of .... cents per cubic yard for each additional .... feet.

12. *Cuts and Fills.*—In cuts the final grade shall be obtained by rolling with a roller weighing not less than five (5) nor more than ten (10) tons. When a fill of one (1) foot or less is required to bring the surface to grade, all vegetable or other perishable matter shall be removed before making the fill.

Embankments shall be formed of earth or other approved materials and shall be constructed in successiv layers, each of which shall extend entirely across the width to be filled. Each layer, which shall not exceed one (1) foot in depth, shall be thoroughly rolled with a roller weighing not less than five (5) nor more than ten (10) tons before the succeeding layer is placed. The roller shall pass over the entire area of each layer of the fill at least twice.



The use of muck, quicksand, soft clay, spongy or perishable material, which will not consolidate under the roller, is prohibited.

When the material excavated from the cuts is not sufficient to make the fills shown on the plans, the contractor shall furnish the necessary extra material to bring the fills to the proper width and graced. When the earthwork is completed the cross-section of the roadbed shall conform to the cross-sectional drawings and profile attached hereto.

All approaches connecting the specified pavement with other streets or alleys intersecting shall also be cut or filled, and secured from settlement, to form a slope of not more than one (1) vertical to ten (10) horizontal, as shown on the profile and plans attached hereto.

III. DRAINAGE.

13. *Drainage.*—The contractor shall construct tile or other drains as shown in the drawings attached hereto. Tile to be laid in a trench at least .... (........) inches wide, and .... (........) feet deep below the established grade of the pavement. Such trench shall be back-

filled with crushed stone or pit run gravel with sand removed, which after light tamping shall be .... (........) inches in depth.

14. *Catch Basins.*—All catch basins and manhole tops and all covers of openings of any kind shall be adjusted to the grade by the contractor at the price shown under this item in his bid.

IV. SUBGRADE.

15. *Construction.*—The subgrade shall be brought to a firm density by rolling the entire area with a self-propelled roller. All portions of the surface of the subgrade which are inaccessible to the roller shall be thoroughly tamped with a hand tamp weighing not less than fifty (50) pounds, the face of which shall not exceed one hundred (100) square inches in area. All soft, spongy, or yielding spots and all vegetable or other perishable matter shall be entirely removed and the space refilled with suitable material.

When the concrete pavement is to be constructed over an old roadbed composed of gravel or macadam, the old roadbed shall be entirely loosened and the material spread for the full width of the roadbed and rolled. All interstices shall be filled with fine material and rolled to make a dense, tight surface.

16. *Acceptance.*—No concrete shall be deposited until the subgrade is checked and accepted by the engineer.

V. FORMS.

17. *Materials.*—Metal or wooden forms shall be free from warp, of sufficient strength to resist springing out of shape, and shall be equal in width to the thickness of the pavement at the edges. Wooden forms shall be of not less than two (2) inch stock. Where the pavement is laid adjacent to buildings, fences or other structures, the side forms may be omitted at the direction of the engineer, but a joint must be constructed as hereinafter specified, between the pavement and such structure.

18. *Setting.*—The forms shall be well staked or otherwise held to the established line and grades, and the upper edge shall conform to the established grade of the alley.

19. *Treatment.*—All mortar and dirt shall be removed from forms before they are used.

VI. PAVEMENT SECTION.

20. *Width, Thickness of Pavement and Cross-Section.*—The width of the pavement shall be .... (........) feet, ... (........) inches, and not less than six (6) inches uniform thickness. In cross-section the finished surface shall have a uniform pitch toward the center line, except that the center twelve (12) inches shall be rounded to the arc of a circle which is tangent to the surface of the concrete at points six (6) inches on each side of the center line, as shown in the plans attached hereto. The dish shall no exceed one-fiftieth (1/50) of the width of the pavement.

VII. JOINTS.

21. *Width and Location.*—Transverse joints shall be one-quarter (¼) inch in width and shall be placed across the pavement perpendicular to the center line, not more than thirty-six (36) feet apart. When the pavement is laid adjacent to buildings or other masonry structures, a joint shall be constructed between the pavement and the structure. All joints shall extend through the entire thickness of the pavement and shall be perpendicular to its surface.

All catch basins, manhole tops, poles or other fixed objects which project through the pavement shall be separated from the concrete by joint filler.

22. *Joint Filler.*—All transverse joints shall be formed by inserting during construction and leaving in place the required thickness of prepared strips of fiber matrix and bitumen, or other similar material of approved quality, which shall extend through the entire thickness of the pavement.

Longitudinal joints shall, at the discretion of the engineer, be formed in the same manner as transverse joints or constructed by filling with bitumen as before specified.

23. *Protected Joints.**—The concrete at transverse joints shall be protected with joint protection plates which shall be rigidly anchored to the concrete. The upper edges of the plates shall be even with each other and the adjoining surface of the concrete. All steel plates varying more than one-quarter (¼) inch from the finished surface of the concrete, as shown on the plans attached hereto, shall be ground to meet the specified requirements, or slabs in which such plates occur shall be removed and replaced with new material by the contractor at his expense.

24. *Unprotected Joints.**—All transverse joints shall extend through the entire thickness of the pavement and the filler shall project not less than one-half (½) inch above the finished surface. Before the pavement is opened to traffic joint filler shall be cut off to the height of one-quarter (¼) inch above the surface of the pavement.

VIII. MEASURING MATERIALS AND MIXING CONCRETE.

25. *Measuring Materials.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 pounds net) shall be considered one (1) cubic foot.

26. *Mixing.*—The materials shall be mixed in a batch mixer approved by the engineer, and irrespective of the size of the batch and rate of speed used, mixing shall continue after all materials are in the drum for at least one (1) minute before any part of the batch is discharged from the drum. The drum shall be completely emptied before receiving material for the succeeding batch. The volume of the mixed material used per batch shall not exceed the manufacturer's rated capacity of the drum in cubic feet of mixed material.

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* When the specification "Protected Joints" is to be used, "Unprotected Joints" should be omitted, and *vice versa*.

27. *Retempering.*—Retempering of mortar or concrete which has partly hardened; that is, remixing with or without additional materials or water, shall not be permitted.

28. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of portland cement to not more than two (2) cubic feet of fine aggregate and not more than three (3) cubic feet of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the coarse aggregate.

A cubic yard of concrete in place shall contain not less than one and seven-tenths (1.7) barrels of cement.

The engineer shall compare the calculated amount of cement required according to these specifications and plans attached hereto with the amounts actually used in each section of concrete between successive transverse joints, as determined by actual count of the number of sacks of cement used in each section. If the amount of cement used in any three (3) adjacent sections (between transverse joints) is less by more than two (2) per cent, or if the amount of cement used in any one section is less by more than five (5) per cent of the amount hereinbefore required, the contractor shall remove all such sections and replace the same with new materials, according to these specifications, at his expense.

29. *Consistency.*—The materials shall be mixed with only sufficient water to produce a concrete which will hold its shape when struck off with a template. The consistency shall not be such as to cause a separation of the mortar from the coarse aggregate in handling.

IX. Reinforcing.

30. *Reinforcing.*—Concrete pavements twenty (20) feet or more in width shall be reinforced. The reinforcement shall have a weight of not less than twenty-eight (28) pounds per one hundred (100) square feet. The ratio of effective areas of reinforcing members at right angles to each other may vary from 1 : 1 to 4 : 1. The spacing between parallel lines of reinforcing members shall not be more than eight (8) inches. A reduction of three (3) pounds from the weight specified shall be allowed for those types of reinforcement not requiring extra metal at intersections.

> NOTE.—The committee is of the opinion that the weight of reinforcement for streets over twenty-five (25) feet wide should be greater than twenty-eight (28) pounds per one hundred (100) square feet.

Reinforcing metal shall not be placed less than two (2) inches from the finished surface of the pavement and otherwise shall be placed as shown on the drawings attached hereto. The reinforcing metal shall extend to within two (2) inches of all joints, but shall not cross them. Adjacent widths of fabric shall be lapped not less than four (4) inches when the lap is made perpendicular to the center line of the pavement and not less than one (1) foot when the lap is made parallel to the center line of the pavement and in most cases the use of reinforcement in pavements sixteen (16) feet wide or over is good practice.

X. PLACING CONCRETE.

31. *Placing Concrete.*—Immediately prior to placing the concrete, the subgrade shall be brought to an even surface. The surface of the subgrade shall be thoroughly wet but shall show no pools of water when the concrete is placed.

After mixing, the concrete shall be deposited rapidly upon the subgrade, to the required depth and for the entire width of the pavement in successive batches, and in a continuous operation without the use of intermediate forms or bulkheads between expansion joints. If the concrete is placed in two courses, as when reinforcement is used, any dirt, sand or dust which collects on the base course shall be removed before the top course is placed. The concrete above the reinforcement shall be placed immediately after mixing and in no case shall more than forty-five (45) minutes elapse between the time that the concrete below the reinforcement has been mixed and the concrete above the reinforcement is placed.

In case of a breakdown concrete shall be mixed by hand to complete the section, or an intermediate transverse joint placed as hereinbefore specified at the point of stopping work. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used.

32. *Finishing.*—The concrete shall be brought to a proper contour by any means which will insure a compact dense surface. Any holes left by removing any material or device used in constructing joints shall be filled immediately with concrete from the latest batch deposited. Concrete adjoining metal protection plates at transverse joints shall be dense in character and shall be given a smooth finish with a steel trowel for a distance of six (6) inches on each side of the joints. The concrete adjacent to unprotected joints shall be finished with a wood float, which is divided through the center and which will permit finishing on both sides of the filler at the same time.

Concrete shall be finished in a manner thoroughly to compact it and produce a surface free from depressions or inequalities of any kind. The finished surface of the pavement shall not vary more than one-quarter ($\frac{1}{4}$) inch from the specified contour.

> NOTE.—It is recommended that the contractor be required at the end of each day's work to stamp in the surface of the concrete with letters one and one-half ($1\frac{1}{2}$) to two (2) inches high and one-quarter ($\frac{1}{4}$) inch deep, the date and his name.

XI. PROTECTION.

33. *Curing and Protection.*—Except as hereinafter specified, the surface of the pavement shall be sprayed with water as soon as the concrete is sufficiently hardened to prevent pitting, and shall be kept wet until an earth or other approved covering is placed. As soon as it can be done without damaging the concrete, the surface of the pavement shall be covered with not less than two (2) inches of earth or other material approved by the engineer, which cover shall be kept wet for at least ten (10) days. When deemed

necessary or advisable by the engineer, freshly laid concrete shall be protected by canvas until such covering can be placed.

Under the most favorable conditions for hardening in hot weather, the pavement shall be closed to traffic for at least fourteen (14) days, and in cool weather for an additional time, to be determined bý the engineer.

When the average temperature is below fifty (50) degrees Fahrenheit, sprinkling and covering of the pavement may be omitted at the discretion of the engineer.

The contractor shall erect and maintain suitable barriers to protect the concrete from traffic and any part of the pavement damaged from traffic or other causes, occurring prior to its official acceptance, shall be repaired or replaced by the contractor at his expense, in a manner satisfactory to the engineer. Before the pavement is thrown open to traffic the covering shall be removed and disposed of as directed by the engineer.

34. *Cold Weather Work.*—Concrete shall not be mixed nor deposited when the temperature is below freezing.

If, at any time during the progress of the work, the temperature is, or in the opinion of the engineer will within twenty-four (24) hours drop to 35 degrees Fahrenheit, the water and aggregates shall be heated, and precautions taken to protect the work from freezing for at least ten (10) days. In no case shall concrete be deposited upon a frozen subgrade.

AMERICAN CONCRETE INSTITUTE.

STANDARD NO. 20.

ADOPTED BY LETTER BALLOT, APRIL 10, 1917.

STANDARD SPECIFICATIONS FOR CONCRETE PAVEMENT BETWEEN STREET CAR TRACKS.

I. Materials.

1. *Cement.*—The cement shall meet the requirements of the Standard Specifications and Tests for Portland Cement, adopted by the American Society for Testing Materials, September 1, 1916, with all subsequent amendments and additions thereto adopted by said Society and by this Institute (Standard No. 1).

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the engineer a fifty (50) pound sample of each of the fine and coarse aggregates proposed for use. These samples shall be tested and if found to pass the requirements of the specifications similar material shall be considered as acceptable for the work. Aggregates containing frost or lumps of frozen material shall not be used.

Fine Aggregate: Fine aggregate shall consist of natural sand or screenings from hard, tough, durable crushed rock or gravel, consisting of quartzite grains or other equally hard material graded from fine to coarse with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes per linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch, and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain vegetable or other deleterious matter nor more than three (3) per cent by weight of clay or loam. Routine field tests shall be made on fine aggregate as delivered. If there is more than seven (7) per cent of clay or loam by volume in one (1) hour's settlement after shaking in an excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement, and three (3) parts fine aggregate, by weight, when made into briquettes, shall show a tensile strength (at seven (7) and twenty-eight (28) days) equal to or greater than the strength of briquettes composed

of one (1) part of the same cement and three (3) parts Standard Ottawa sand by weight. The percentage of water used in making the briquettes of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquettes of Standard consistency. In other respects all briquettes shall be made in accordance with the methods outlined in the Standard Specifications and Tests for Portland Cement, adopted by the American Society for Testing Materials, September 1, 1916.

Coarse Aggregate: Coarse aggregate shall consist of clean, hard, tough, durable crushed granite or trap rock graded in size, free from vegetable or other deleterious matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall be such as to pass a two (2) inch round opening and shall range from two (2) inches down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

3. *Mixed Aggregate.*—Crusher run stone, bank run gravel or artificially prepared mixtures of fine and coarse aggregate shall not be used.

4. *Water.*—Water shall be clean, free from oil, acid, alkali or vegetable matter.

5. *Joint Filler.*—The filler for joints shall consist of prepared strips of fiber matrix and bitumen, or similar material of approved quality one-quarter ($\frac{1}{4}$) inch in thickness. The width of the joint filler shall be at least one-half ($\frac{1}{2}$) inch greater than the thickness of the pavement at any point. Prior to submitting bid, the contractor shall obtain approval of the engineer for the joint filler which he proposes to use.

6. *Bitumen.*—Pitch or asphalt used for breaking of bond between sections of concrete as specified shall be of such a character as to firmly adhere to the surface of the concrete base and remain in position when solidified.

II. PREPARATION OF SURFACE OF CONCRETE BASE.

7. *Placing Bitumen.*—After the concrete base has hardened the entire area upon which the concrete pavement is to be placed shall be completely covered and brought to an even surface by spreading thereon a mat of bitumen and sand.

The bitumen shall be heated in a kettle which will insure a uniform heat throughout its contents.

As the bitumen is needed it shall be carried directly to the work and deposited evenly over the surface of the concrete base. Each square yard of surface shall be covered with at least one-third ($\frac{1}{3}$) gallon, and not more than one-half ($\frac{1}{2}$) gallon of bitumen.

8. *Placing Sand.*—Immediately after the bitumen is deposited it shall be completely covered with sand.

After the bitumen has solidified the surplus sand shall be broomed from the surface.

Care should be exercised so that no bitumen will come in contact with the rails above the rail base except as hereinafter specified.

9. *Old Track.*—Where an old track base is to be prepared for concrete

pavement, sufficient bitumen and sand shall be used to secure an even surface. Any depressions over one (1) inch in depth shall be filled with mortar or concrete which shall be allowed to harden before applying bitumen.

III. FORMS.

10. *Materials.*—Where forms are required, they shall be free from warp and of sufficient strength to resist springing out of shape. Wooden forms shall be of not less than two (2) inch stock.

11. *Setting.*—The forms shall be well staked or otherwise held to the established line and grades.

12. *Treatment.*—All mortar and dirt shall be removed from the forms before they are used.

IV. PAVEMENT SECTION.

13. *In Track.*—The concrete shall extend from web to web of rail.

When grooved rails are used the concrete surface at rail shall be level with the top of the lip of rail and between rails conform to a straightedge placed from lip to lip of rail.

When T-rails are used the concrete surface at rails shall be one and one-eighth ($1\frac{1}{8}$) inches below the top of rail. At the center line of track the surface of the concrete shall be $\frac{1}{4}$ inch below the top of the T-rails.

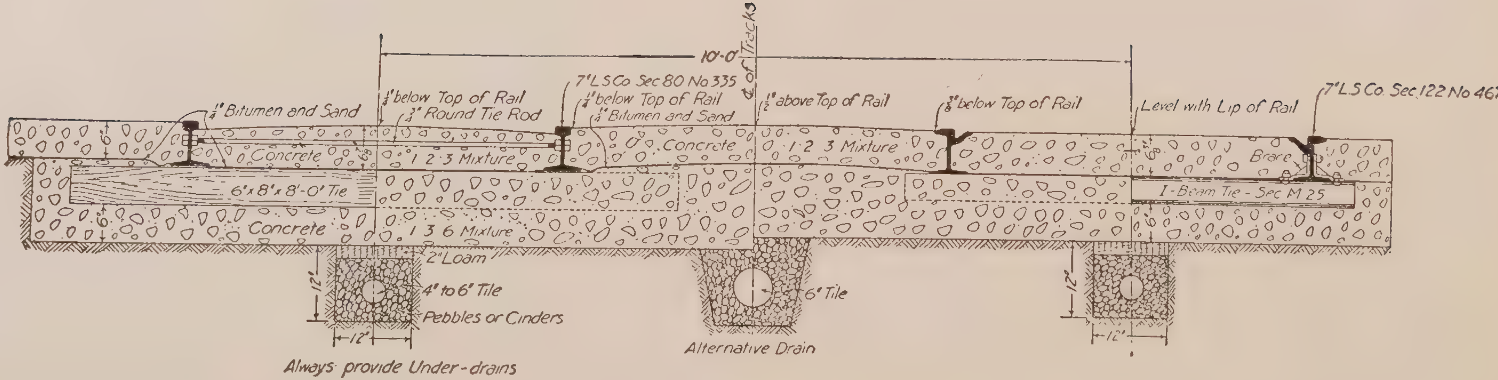
14. *Between Track.*—The concrete shall extend from web to web of rails and have a thickness such that the surface of the concrete shall be one-quarter ($\frac{1}{4}$) inch below the top of T-rails and three-eighths ($\frac{3}{8}$) inch below the top of grooved rails at rails and crowned at right angles to the center line of tracks in an arc of a circle passing through a point on the center line between tracks one-half ($\frac{1}{2}$) inch above the top of rail.

V. JOINTS.

15. *Location of Joints.*—When necessary to continue traffic over one track of double track construction a longitudinal joint shall be constructed on the center line between tracks.

When the concrete street pavement cannot be laid at the same time as the pavement is being placed on the car tracks and it is necessary to operate over the track while the street pavement is being laid, then the street and car track pavements shall be completely separated by a one-quarter ($\frac{1}{4}$) inch longitudinal joint which shall be placed at the outside edge of the concrete track base. When the concrete pavement is laid at the same time as the track is paved, the street pavement should be continuous without longitudinal joints to the outside of the rail. The bond between the rail and concrete at this point, however, shall be broken by painting the outside of the rail with bitumen.

16. *Joint Filler.*—All longitudinal joints shall be formed by inserting during construction and leaving in place the required thickness of prepared strips of fiber matrix and bitumen or similar material of approved quality which shall extend through the entire thickness of the pavement.



DESIGN
FOR
CONCRETE PAVEMENT
BETWEEN
STREET CAR TRACKS

17. *Longitudinal Joints.*—All longitudinal joints shall extend through the entire thickness of the pavement and the filler shall project not less than one-half ($\frac{1}{2}$) inch above the finished surface. Before the pavement is open to traffic joint filler shall be cut off to a height of one-quarter ($\frac{1}{4}$) inch above the surface of the pavement.

18. *Construction Joint.*—When necessary to discontinue the placing of concrete for a period of more than thirty (30) minutes, the concrete should be brought to a straight perpendicular edge at right angles to the center line of track by means of a bulkhead form.

When construction is resumed the form shall be removed and the concrete placed flush against the section formerly placed.

VI. MEASURING MATERIALS AND MIXING CONCRETE.

19. *Measuring Materials.*—The method of measuring the materials for the concrete, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 pounds net) shall be considered one (1) cubic foot.

20. *Mixing.*—The materials shall be mixed in a batch mixer approved by the engineer and irrespective of the size of the batch and rate of speed used mixing shall continue after all materials are in the drum for at least one (1) minute before any part of the batch is discharged from the drum. The drum shall be completely emptied before receiving material for the succeeding batch. The volume of the mixed material used per batch shall not exceed the manufacturer's rated capacity of the drum in cubic feet of mixed material.

21. *Retempering.*—Retempering of mortar or concrete which has partly hardened; that is, remixing with or without additional materials or water, shall not be permitted.

22. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of Portland cement to not more than two (2) cubic feet of fine aggregate and not more than three (3) cubic feet of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half ($\frac{1}{2}$) the volume of the coarse aggregate.

23. *Cement Required.*—A cubic yard of concrete in place shall contain not less than one and seven-tenths (1.7) barrels of cement.

The engineer shall compare the calculated amount of cement required according to these specifications and plans attached hereto with the amounts actually used in the concrete between sections containing twenty (20) cubic yards of concrete, as determined by actual count of the number of sacks of cement used in each section. If the amount of cement used in any three (3) adjacent sections is less by more than two (2) per cent, or if the amount of cement used in any one section is less by more than five (5) per cent of the amount hereinbefore required the contractor shall remove all such sections and replace the same with new materials, according to these specifications, at his expense.

24. *Consistency.*—The materials for the pavement shall be mixed with

only sufficient water to produce a concrete which will hold its shape when struck off with a template. The consistency shall not be such as to cause a separation of the mortar from the coarse aggregate in handling.

VII. PLACING CONCRETE.

25. *Placing Concrete.*—After mixing, the concrete shall be deposited rapidly upon the finished bitumen and sand cushion to the required depth and for the entire width of the section to be concreted in successive batches and in a continuous operation. The concrete shall be thoroughly tamped until surplus mortar is brought to the surface. A foundryman's peen or similar tool shall be used to thoroughly compact the concrete against the rails, rail fastenings and tie rods.

26. *Finishing.*—The surface of the concrete shall be brought to a proper contour by any means which will insure a compact dense surface. Any holes left by removing any material or device used in constructing joints shall be filled immediately with concrete from the latest batch deposited.

The concrete adjacent to longitudinal joints where a joint filler is required shall be finished with a wood float, which is divided through the center and which will permit finishing of the car track at the same elevation as the adjoining street pavement.

The finished surface of the pavement shall not vary more than one-quarter ($\frac{1}{4}$) inch from the true shape when measured by a template resting upon the top of rails and at right angles to the center line of track. The surface of the pavement for a distance six (6) inches each side of the rails shall be finished to dimensions exactly as hereinbefore specified; no variations either way will be permitted.

After the concrete has been struck off, workmen shall not be permitted upon it until thoroughly hardened.

VIII. PROTECTION.

27. *Curing and Protection.*—Except as hereinafter specified, the surface of the pavement shall be sprayed with water as soon as the concrete is sufficiently hardened to prevent pitting, and shall be kept wet until an earth or other approved covering is placed. As soon as it can be done without damaging the concrete the surface of the pavement shall be covered with not less than two (2) inches of earth or other material approved by the engineer, which cover shall be kept wet for at least ten (10) days. When deemed necessary or advisable by the engineer, freshly laid concrete shall be protected by canvas until such covering can be placed.

Under the most favorable conditions for hardening in hot weather the pavement shall be closed to traffic for at least fourteen (14) days, and in cool weather for an additional time, to be determined by the engineer.

When the average temperature is below fifty (50) degrees Fahrenheit, sprinkling and covering of the pavement may be omitted at the discretion of the engineer.

The contractor shall erect and maintain suitable barriers to protect the concrete from traffic, both rail and vehicle, and any part of the pavement damaged from traffic or other causes, occurring prior to its official acceptance, shall be repaired or replaced by the contractor at his expense, in a manner satisfactory to the engineer. Before the pavement is thrown open to traffic, the covering shall be removed and disposed of as directed by the engineer.

28. *Cold Weather Work.*—Concrete shall not be mixed nor deposited when the temperature is below freezing.

If at any time during the progress of the work the temperature is, or in the opinion of the engineer will within twenty-four (24) hours drop to thirty-five (35) degrees Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least ten (10) days. In no case shall concrete be deposited upon a frozen subgrade.

AMERICAN CONCRETE INSTITUTE.

STANDARD NO. 10.

ADOPTED BY LETTER BALLOT, APRIL 10, 1917.

STANDARD SPECIFICATIONS AND BUILDING REGULATIONS FOR THE MANUFACTURE AND USE OF CONCRETE ARCHITECTURAL STONE, BUILDING BLOCKS, AND BRICK.

1. Concrete architectural stone and building blocks for solid or hollow walls and concrete brick made in accordance with the following specifications and meeting the requirements thereof may be used in building construction.

2. *Tests.*—Concrete architectural stone, building blocks for hollow and solid walls and concrete brick must be subjected to (*a*) Compression and (*b*) Absorption tests. All tests must be made in a testing laboratory of recognized standing.

3. *Ultimate Compressive Strength.*—(*a*) Solid concrete stone, building blocks and brick. In the case of solid stone, blocks, and brick, the ultimate compressive strength at 28 days must average not less than fifteen hundred (1,500) lb. per sq. in. of gross cross-sectional area of the stone as used in the wall and must not fall below one thousand (1,000) lb. per sq. in. in any test.

(*b*) Hollow and two piece building blocks. The ultimate compressive strength of hollow and two piece building blocks at 28 days must average one thousand (1,000) lb. per sq. in. of gross cross-sectional area of the block as used in the wall, and must not fall below seven hundred (700) lb. per sq. in. in any test.

4. *Gross Cross-Sectional Areas.*—(*a*) Solid concrete stone, blocks, and brick. The cross-sectional area shall be considered as the minimum area in compression.

(*b*) Hollow building blocks. In the case of hollow building blocks, the gross cross-sectional area shall be considered as the product of the length by the width of the block. No allowance shall be made for the air space of the block.

(*c*) Two piece building blocks. In the case of two piece building blocks, if only one block is tested at a time, the gross cross-sectional area shall be regarded as the product of the length of the block by one-half of the width of the wall for which the block is intended. If two blocks are tested together,

then the gross cross-sectional area shall be regarded as the product of the length of the block by the full width of the wall for which the block is intended.

5. *Absorption.*—The absorption at 28 days (being the weight of the water absorbed divided by the weight of the dry sample) must not exceed ten (10) per cent when tested as hereinafter specified.

6. *Samples.*—At least six samples must be provided for the purpose of testing. Such samples must represent the ordinary commercial product. In cases where the material is made and used in special shapes and forms too large for testing in the ordinary machine, smaller specimens shall be used as may be directed. Whenever possible, the tests shall be made on full sized samples.

7. *Compression Tests.*—Compression tests shall be made as follows: The sample to be tested must be carefully measured and then bedded in plaster of paris or other cementitious material in order to secure uniform bearing in the testing machine. It shall then be loaded to failure. The compressive strength in pounds per square inch of gross cross-sectional area shall be regarded as the quotient obtained by dividing the total applied load in pounds by the gross cross-sectional area, which area shall be expressed in square inches computed according to article 4.

When such tests must be made on cut sections of blocks, the pieces of the block must first be carefully measured. The samples shall then be bedded to secure uniform bearing, and loaded to failure. In this case, however, the compressive strength in pounds per square inch of net area must be obtained and the net area shall be regarded as the minimum bearing area in compression. The average of the compressive strength of the two portions of blocks shall be regarded as the compressive strength of the samples submitted. This net compressive strength shall then be reduced to compressive strength in pounds per square inch of gross cross-sectional area as follows:

The net area of a full sized block shall be carefully calculated and the total compressive strength of the block will be obtained by multiplying this area by the net compressive strength obtained above. This total gross compressive strength shall be divided by the gross cross-sectional area as figured by article 4 to obtain the compressive strength in pounds per square inch of gross cross-sectional area.

When testing other than rectangular blocks, great care must be taken to apply the load at the center of the gravity of the specimen.

8. *Absorption Tests.*—The samples shall be first thoroughly dried to a constant weight at a temperature not to exceed two hundred and twelve (212) degrees Fahrenheit, and the weight recorded. After drying, the sample shall be immersed in clean water for a period of forty-eight hours. The sample shall then be removed; the surface water wiped off, and the sample re-weighed. The percentage of absorption shall be regarded as the weight of the water absorbed divided by the weight of the dry sample multiplied by one hundred (100).

9. *Limit of Loading.*—(*a*) Hollow walls of concrete building blocks. The load on any hollow walls of concrete blocks, including the superimposed weight

of the wall, shall not exceed one hundred and sixty-seven (167) lb. per sq. in. of gross area. If the floor loads are carried on girders or joists resting on cement pilasters filled in place with slush concrete mixed in proportion of one (1) part cement, not to exceed two (2) parts of sand and four (4) parts of gravel or crushed stone, said pilasters may be loaded not to exceed three hundred (300) lb. per sq. in. of gross cross-sectional area.

(*b*) Solid walls of concrete blocks. Solid walls built of architectural stone, blocks or brick and laid in portland cement mortar or hollow block walls filled with concrete shall not be loaded to exceed three hundred (300) lb. per sq. in. of gross cross-sectional area.

10. *Girders and Joists.*—Wherever girders or joists rest upon walls in such a manner as to cause concentrated loads of over four thousand (4,000) lb. the blocks supporting the girders or joists must be made solid for at least eight (8) in. from the inside face of the wall, except where a suitable bearing plate is provided to distribute the load over a sufficient area to reduce the stress so it will conform to the requirements of article 9.

When the combined live and dead floor loads exceed sixty (60) lb. per sq. ft., the floor joists shall rest on a steel plate not less than three-eighths ($\frac{3}{8}$) of an inch thick and of a width one-half to one inch less than the wall thickness. In lieu of said steel plate the joists may rest on a solid block which may be three (3) or four (4) in. less in wall thickness than the building wall, except in instances where the wall is eight (8) in. thick, in which cases the solid blocks shall be the same thickness as the building wall.

11. *Thickness of Walls.*—(*a*) Thickness of bearing walls shall be such as will conform to the limit of loading given in article 9. In no instance shall bearing walls be less than eight (8) in. thick. Hollow walls eight (8) in. thick shall not be over sixteen (16) ft. high for one story or more than a total of twenty-four (24) ft. for two stories.

(*b*) Walls of residence and buildings commonly known as apartment buildings not exceeding four stories in height, in which the dead floor load does not exceed sixty (60) lb. or the live load (60) lb. per sq. ft., shall have a minimum thickness in inches as shown in Table I.

Table I.

No. of Stories.	Basement, in.	First Story, in.	Second Story, in.	Third Story, in.	Fourth Story, in.
1	8	8	..	..	..
2	10	8	8	..	..
3	12	12	10	8	..
4	16	12	12	10	8

12. *Variation in Thickness of Walls.*—(*a*) Wherever walls are decreased in thickness the top course of the thicker wall shall afford a solid bearing for the webs or walls of the course of the concrete block above.

13. *Bonding and Bearing Walls.*—Where the face wall is constructed of both hollow concrete blocks and brick, the facing shall be bonded into the

backing, either with headers projecting four (4) in. into the brick work, every fourth course being a header course, or with approved ties, no brick backing to be less than eight (8) in. thick. Where the walls are made entirely of concrete blocks, but where said blocks have not the same width as the wall, every fifth course shall overlap the course below by not less than four (4) in. unless the wall system alternates the cross bond through the wall in each course.

14. *Curtain Walls.*—For curtain walls the limit of loading shall be the same as given in article 9. In no instance shall curtain walls be less than eight (8) in. in thickness.

15. *Party Walls.*—Walls of hollow concrete blocks used in the construction of party walls shall be filled in place with concrete in the proportion and manner described in article 9.

16. *Partition Walls.*—Hollow partition walls of concrete blocks may be of the same thickness as required in hollow tile, terra cotta or plaster blocks for like purposes.

AMERICAN CONCRETE INSTITUTE.

STANDARD NO. 21.

ADOPTED BY LETTER BALLOT, APRIL 10, 1917.

STANDARD SPECIFICATIONS FOR REINFORCED-CONCRETE FENCE POSTS.

1. These specifications cover the manufacture of reinforced-concrete line posts made by hand processes or by machine.

2. The acceptability of reinforced-concrete fence posts under these specifications shall be determined, (1) by the results of load tests on representative finished posts, and (2) by measurements and visual inspection to determine whether the posts conform to the required dimensions and shape, and are free from visible defects.

MATERIALS AND MANUFACTURE.

3. *Concrete.*—By concrete is meant suitable proportions of portland cement, hard, durable mineral aggregate and water, which are properly mixed and allowed to harden under favorable conditions.

4. *Reinforcement.*—The posts shall be reinforced by means of rods, wires, or other shapes of steel, iron or other metal, suitably disposed to develop the necessary resistance to tensile stresses. Reinforcement shall be free from scaly rust, scale paint or coatings of any character which will tend to destroy the bond.

5. *Cement.*—The cement shall meet the requirements of the current Standard Specifications and Tests for Portland Cement of the American Society for Testing Materials. (Am. Conc. Inst., Standard No. 1).

6. *Fine Aggregate.*—Fine aggregate shall consist of natural sand or screenings from hard, tough, durable crushed rock or gravel consisting of quartzitic grains or other equally hard material graded from coarse to fine with the coarse particles predominating. Fine aggregate, when dry, shall pass a sieve having 4 meshes per linear inch; not more than thirty (30) per cent shall be finer than a sieve having fifty (50) meshes per linear inch, and not more than eight (8) per cent shall be finer than a sieve having one hundred (100) meshes per linear inch. The above mentioned percentages shall be computed on the basis of weight. Fine aggregate shall contain no vegetable or other deleterious materials, and not more than three (3) per cent, by weight, of clay or loam.

Fine aggregates which give a mortar strength equal to or higher than the minimum value at any of the ages named below, shall be considered as fulfilling the mortar strength requirements of this specification.

Age at Test.	Minimum Strength of 1-3 Fine Aggregate Matter.
72 hours	1.00 times (A)
7 days	.90 times (A)
28 days	.80 times (A)

(A) equals the strength of 1–3 standard Ottawa sand mortar specimens of same form and size, of similar plasticity, made by the same operator using the same cement.

The tests shall be made on mortars composed of one (1) part portland cement and three (3) parts, by weight, of fine aggregate or standard Ottawa sand. The test specimen shall be made, stored, and tested in the same manner. All mortar strength tests shall be made under laboratory conditions in accordance with recognized standards. Each value shall be the average from tests of not fewer than three (3) specimens.

7. *Coarse Aggregate.*—Coarse aggregate shall consist of clean tough, durable pebbles or crushed rock of graded sizes, free from vegetable or other deleterious matter, and shall contain no soft, flat, or elongated particles. The maximum size of the coarse aggregate shall not be greater than one-fifth ($\frac{1}{5}$) the smallest dimensions of the post. Not more than ten (10) per cent shall pass a screen having four (4) square meshes per linear inch.

8. *Water.*—Water shall be clean, free from oil, acid, alkali, or vegetable matter.

PROPORTIONING AND MIXING CONCRETE.

9. *Measuring Materials.*—The method of measuring the cement, aggregate, and water shall be one which will insure separate and uniform proportions of each of the materials in each batch of concrete. A sack of portland cement (94 lb. net) shall be considered equivalent to one (1) cubic foot.

10. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of Portland cement to a volume of aggregate, not greater than that given below. The smallest sieve which passes seventy-five (75) per cent or more of fine aggregate or combined fine and coarse aggregate, shall be considered to govern in determining the maximum quantity of aggregate which shall be mixed with one (1) sack of portland cement.

Graded Aggregate.	Proportions.	
	Cement, sack.	Max. Volume of Aggregate, cu. ft.
From finest particles up to $\frac{3}{4}$ in. square opening	1	$4\frac{1}{2}$
From finest particles up to $\frac{1}{2}$ in. square opening	1	$4\frac{1}{4}$
From finest particles up to $\frac{3}{8}$ in. square opening	1	4
From finest particles up to No. 4 sieve opening	1	$3\frac{1}{2}$
From finest particles up to No. 8 sieve opening	1	3
From finest particles up to No. 14 sieve opening	1	$2\frac{1}{2}$
From finest particles up to No. 28 sieve opening	1	2

The manufacturer may use leaner proportions than these shown in the table, provided posts meet the test requirements of Article 16 and providing also that test cylinders 6 in. in diameter and 12 in. long made of the concrete

as mixed for the posts, show a compressive strength of not less than 2,000 lb. per sq. in. at the age of 28 days.

11. *Consistency.*—The concrete shall be mixed with the least quantity of water which can be used and obtain a workable concrete.

12. *Mixing.*—The ingredients of the concrete shall be mixed until they are thoroughly incorporated into a mass.

REINFORCEMENT.

13. *Placing Reinforcement.*—The reinforcement shall consist of not fewer than three (3) bars, rods, wires, or other suitable shapes. The reinforcement shall be placed in such a manner as to insure its remaining in the desired position during the placing and hardening of the concrete. At no point shall the reinforcement be closer than one-half ($\frac{1}{2}$) inch to the surface of the finished post.

14. *End Anchorage.*—Reinforcing members composed of wire which has been cold-drawn, galvanized, or treated in similar manner prior to use must be provided with an end loop or other form of end anchorage.

15. *Wire Fasteners.*—If means for the fence attachment are provided in the manufacture of the post, the fastener must be of non-corroding material, and suitable for this purpose.

TESTS OF REINFORCED CONCRETE POSTS.

16. *Test Requirements.*—At any time subsequent to the age at which the posts are offered for use, they shall show an ultimate breaking load of one thousand (1000) lb. or more when placed in a horizontal position on rigid supports three (3) inches from the ends and loaded with a concentrated load applied in the plane of the ground line. One-half the number of posts tested shall be tested with wire face downward and one-half with wire face upward.

17. *Payment for Tests.*—The manufacturer shall furnish without additional charge not more than one (1) post for each one hundred (100) posts furnished, but in no case, fewer than five posts, for testing purposes. Posts tested in excess of the larger of these quantities shall be paid for by the purchaser at the contract price; if a smaller number of posts is tested, only those tested shall be exempt from payment. Full report of these tests shall be furnished to the manufacturer, or the seller on request. The expense of making all tests on posts shall be borne by the purchaser.

VISUAL INSPECTION.

18. All fence posts shall be given a thorough visual inspection by a competent inspector employed by the purchaser. All posts shall be straight, true to dimensions and of a workmanlike finish. Posts which are clearly faulty, or have been broken or otherwise structurally injured during manufacture or storage, shall not be accepted.

19. The manufacturer or other seller of posts shall afford the inspector all reasonable facilities for sampling of materials and for inspecting all processes of manufacture, and for proper visual inspection and tests of the finished posts.

STANDARD SPECIFICATIONS OF THE AMERICAN SOCIETY FOR TESTING MATERIALS

ADOPTED BY THE

AMERICAN CONCRETE INSTITUTE.

At the annual convention of the American Concrete Institute, February, 1917, two standards of the American Society for Testing Materials were adopted as standards by the American Concrete Institute. They are reprinted, as follows, on pp. 472–508.

Standard No. 1—Portland Cement.

Standard No. 9—Drain Tile.

AMERICAN SOCIETY FOR TESTING MATERIALS

PHILADELPHIA, PA., U. S. A.

AFFILIATED WITH THE

INTERNATIONAL ASSOCIATION FOR TESTING MATERIALS.

STANDARD SPECIFICATIONS AND TESTS FOR PORTLAND CEMENT.[1]

These specifications are the result of several years' work of a special committee representing a United States Government Departmental Committee, the Board of Direction of the American Society of Civil Engineers, and Committee C-1 on Cement of the American Society for Testing Materials.

Serial Designation: C 9 – 17.

The specifications and tests for this material are issued under the fixed designation C 9; the final number indicates the year of original issue, or in the case of revision, the year of last revision.

ADOPTED, 1904; REVISED, 1908, 1909, 1916.

SPECIFICATIONS.

Definition. 1. Portland cement is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion, an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

I. CHEMICAL PROPERTIES.

Chemical Limits. 2. The following limits shall not be exceeded:

Loss on ignition, per cent	4.00
Insoluble residue, per cent	0.85
Sulfuric anhydride (SO_3), per cent	2.00
Magnesia (MgO), per cent	5.00

[1] These specifications and tests were adopted by letter ballot of the Society on September 1, 1916, with the understanding that they will not become effective till January 1, 1917.

II. PHYSICAL PROPERTIES.

3. The specific gravity of cement shall be not less than 3.10 (3.07 for white Portland cement). Should the test of cement as received fall below this requirement a second test may be made upon an ignited sample. The specific gravity test will not be made unless specifically ordered. **Specific Gravity.**

4. The residue on a standard No. 200 sieve shall not exceed 22 per cent by weight. **Fineness.**

5. A pat of neat cement shall remain firm and hard, and show no signs of distortion, cracking, checking, or disintegration in the steam test for soundness. **Soundness.**

6. The cement shall not develop initial set in less than 45 minutes when the Vicat needle is used or 60 minutes when the Gillmore needle is used. Final set shall be attained within 10 hours. **Time of Setting.**

7. The average tensile strength in pounds per square inch of not less than three standard mortar briquettes (see Section 51) composed of one part cement and three parts standard sand, by weight, shall be equal to or higher than the following: **Tensile Strength.**

Age at Test, days.	Storage of Briquettes.	Tensile Strength lb. per sq. in.
7	1 day in moist air, 6 days in water..................................	200
28	1 day in moist air, 27 days in water................................	300

8. The average tensile strength of standard mortar at 28 days shall be higher than the strength at 7 days.

III. PACKAGES, MARKING AND STORAGE.

9. The cement shall be delivered in suitable bags or barrels with the brand and name of the manufacturer plainly marked thereon, unless shipped in bulk. A bag shall contain 94 lb. net. A barrel shall contain 376 lb. net. **Packages and Marking.**

10. The cement shall be stored in such a manner as to permit easy access for proper inspection and identification of each shipment, and in a suitable weather-tight building which will protect the cement from dampness. **Storage.**

IV. INSPECTION.

Inspection.

11. Every facility shall be provided the purchaser for careful sampling and inspection at either the mill or at the site of the work, as may be specified by the purchaser. At least 10 days from the time of sampling shall be allowed for the completion of the 7-day test, and at least 31 days shall be allowed for the completion of the 28-day test. The cement shall be tested in accordance with the methods hereinafter prescribed. The 28-day test shall be waived only when specifically so ordered.

V. REJECTION.

Rejection.

12. The cement may be rejected if it fails to meet any of the requirements of these specifications.

13. Cement shall not be rejected on account of failure to meet the fineness requirement if upon retest after drying at 100° C. for one hour it meets this requirement.

14. Cement failing to meet the test for soundness in steam may be accepted if it passes a retest using a new sample at any time within 28 days thereafter.

15. Packages varying more than 5 per cent from the specified weight may be rejected; and if the average weight of packages in any shipment, as shown by weighing 50 packages taken at random, is less than that specified, the entire shipment may be rejected.

TESTS.

VI. SAMPLING.

Number of Samples.

16. Tests may be made on individual or composite samples as may be ordered. Each test sample should weigh at least 8 lb.

17. (*a*) *Individual Sample.*—If sampled in cars one test sample shall be taken from each 50 bbl. or fraction thereof. If sampled in bins one sample shall be taken from each 100 bbl.

(*b*) *Composite Sample.*—If sampled in cars one sample shall be taken from one sack in each 40 sacks (or 1 bbl. in each 10 bbl.) and combined to form one test sample. If sampled in bins or warehouses one test sample shall represent not more than 200 bbl.

18. Cement may be sampled at the mill by any of the following methods that may be practicable, as ordered: **Method of Sampling.**

(*a*) *From the Conveyor Delivering to the Bin.*—At least 8 lb. of cement shall be taken from approximately each 100 bbl. passing over the conveyor.

(*b*) *From Filled Bins by Means of Proper Sampling Tubes.*—Tubes inserted vertically may be used for sampling cement to a maximum depth of 10 ft. Tubes inserted horizontally may be used where the construction of the bin permits. Samples shall be taken from points well distributed over the face of the bin.

(*c*) *From Filled Bins at Points of Discharge.*—Sufficient cement shall be drawn from the discharge openings to obtain samples representative of the cement contained in the bin, as determined by the appearance at the discharge openings of indicators placed on the surface of the cement directly above these openings before drawing of the cement is started.

19. Samples preferably shall be shipped and stored in air-tight containers. Samples shall be passed through a sieve having 20 meshes per linear inch in order to thoroughly mix the sample, break up lumps and remove foreign materials. **Treatment of Sample.**

VII. CHEMICAL ANALYSIS.

LOSS ON IGNITION.

20. One gram of cement shall be heated in a weighed covered platinum crucible, of 20 to 25-cc. capacity, as follows, using either method (*a*) or (*b*) as ordered: **Method.**

(*a*) The crucible shall be placed in a hole in an asbestos board, clamped horizontally so that about three-fifths of the crucible projects below, and blasted at a full red heat for 15 minutes with an inclined flame; the loss in weight shall be checked by a second blasting for 5 minutes. Care shall be taken to wipe off particles of asbestos that may adhere to the crucible when withdrawn from the hole in the board. Greater neatness and shortening of the time of heating are secured by making a hole to fit the crucible in a circular disk of sheet platinum and placing this disk over a somewhat larger hole in an asbestos board.

(*b*) The crucible shall be placed in a muffle at any tempera-

ture between 900 and 1000° C. for 15 minutes and the loss in weight shall be checked by a second heating for 5 minutes.

Permissible Variation. 21. A permissible variation of 0.25 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 4 per cent.

INSOLUBLE RESIDUE.

Method. 22. To a 1-g. sample of cement shall be added 10 cc. of water and 5 cc. of concentrated hydrochloric acid; the liquid shall be warmed until effervescence ceases. The solution shall be diluted to 50 cc. and digested on a steam bath or hot plate until it is evident that decomposition of the cement is complete. The residue shall be filtered, washed with cold water, and the filter paper and contents digested in about 30 cc. of a 5-per-cent solution of sodium carbonate, the liquid being held at a temperature just short of boiling for 15 minutes. The remaining residue shall be filtered, washed with cold water, then with a few drops of hot hydrochloric acid, 1 : 9, and finally with hot water, and then ignited at a red heat and weighed as the insoluble residue.

Permissible Variation. 23. A permissible variation of 0.15 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 0.85 per cent.

SULFURIC ANHYDRIDE.

Method. 24. One gram of the cement shall be dissolved in 5 cc. of concentrated hydrochloric acid diluted with 5 cc. of water, with gentle warming; when solution is complete 40 cc. of water shall be added, the solution filtered, and the residue washed thoroughly with water. The solution shall be diluted to 250 cc., heated to boiling and 10 cc. of a hot 10-per-cent solution of barium chloride shall be added slowly, drop by drop, from a pipette and the boiling continued until the precipitate is well formed. The solution shall be digested on the steam bath until the precipitate has settled. The precipitate shall be filtered, washed, and the paper and contents placed in a weighed platinum crucible and the paper slowly charred and consumed without flaming. The barium sulfate shall then be ignited and weighed. The weight obtained multiplied by 34.3 gives the percentage of sulfuric anhydride. The acid filtrate obtained in

the determination of the insoluble residue may be used for the estimation of sulfuric anhydride instead of using a separate sample.

25. A permissible variation of 0.10 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 2.00 per cent. **Permissible Variation.**

MAGNESIA.

26. To 0.5 g. of the cement in an evaporating dish shall be added 10 cc. of water to prevent lumping and then 10 cc. of concentrated hydrochloric acid. The liquid shall be gently heated and agitated until attack is complete. The solution shall then be evaporated to complete dryness on a steam or water bath. To hasten dehydration the residue may be heated to 150 or even 200° C. for one-half to one hour. The residue shall be treated with 10 cc. of concentrated hydrochloric acid diluted with an equal amount of water. The dish shall be covered and the solution digested for ten minutes on a steam bath or water bath. The diluted solution shall be filtered and the separated silica washed thoroughly with water.[1] Five cubic centimeters of concentrated hydrochloric acid and sufficient bromine water to precipitate any manganese which may be present, shall be added to the filtrate (about 250 cc.). This shall be made alkaline with ammonium hydroxide, boiled until there is but a faint odor of ammonia, and the precipitated iron and aluminum hydroxides, after settling, shall be washed with hot water, once by decantation and slightly on the filter. Setting aside the filtrate, the precipitate shall be transferred by a jet of hot water to the precipitating vessel and dissolved in 10 cc. of hot hydrochloric acid. The paper shall be extracted with acid, the solution and washings being added to the main solution. The aluminum and iron shall then be reprecipitated at boiling heat by ammonium hydroxide and bromine water in a volume of about 100 cc., and the second precipitate shall be collected and washed on the filter used in the first instance if this is still intact. To the combined filtrates from the hydroxides of iron and aluminum, reduced in volume if need be, 1 cc. of ammonium hydroxide shall be added, the solution brought **Method.**

[1] Since this procedure does not involve the determination of silica, a second evaporation is unnecessary.

to boiling, 25 cc. of a saturated solution of boiling ammonium oxalate added, and the boiling continued until the precipitated calcium oxalate has assumed a well-defined granular form. The precipitate after one hour shall be filtered and washed, then with the filter shall be placed wet in a platinum crucible, and the paper burned off over a small flame of a Bunsen burner; after ignition it shall be redissolved in hydrochloric acid and the solution diluted to 100 cc. Ammonia shall be added in slight excess, and the liquid boiled. The lime shall then be reprecipitated by ammonium oxalate, allowed to stand until settled, filtered and washed. The combined filtrates from the calcium precipitates shall be acidified with hydrochloric acid, concentrated on the steam bath to about 150 cc., and made slightly alkaline with ammonium hydroxide, boiled and filtered (to remove a little aluminum and iron and perhaps calcium). When cool, 10 cc. of saturated solution of sodium-ammonium-hydrogen phosphate shall be added with constant stirring. When the crystallin ammonium-magnesium orthophosphate has formed, ammonia shall be added in moderate excess. The solution shall be set aside for several hours in a cool place, filtered and washed with water containing 2.5 per cent of NH_3. The precipitate shall be dissolved in a small quantity of hot hydrochloric acid, the solution diluted to about 100 cc., 1 cc. of a saturated solution of sodium-ammonium-hydrogen phosphate added, and ammonia drop by drop, with constant stirring, until the precipitate is again formed as described and the ammonia is in moderate excess. The precipitate shall then be allowed to stand about two hours, filtered and washed as before. The paper and contents shall be placed in a weighed platinum crucible, the paper slowly charred, and the resulting carbon carefully burned off. The precipitate shall then be ignited to constant weight over a Meker burner, or a blast not strong enough to soften or melt the pyrophosphate. The weight of magnesium pyrophosphate obtained multiplied by 72.5 gives the percentage of magnesia. The precipitate so obtained always contains some calcium and usually small quantities of iron, aluminum, and manganese as phosphates.

Permissible Variation.

27. A permissible variation of 0.4 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 5.00 per cent.

VIII. DETERMINATION OF SPECIFIC GRAVITY.

28. The determination of specific gravity shall be made with a standardized Le Chatelier apparatus which conforms to the requirements illustrated in Fig. 1. This apparatus is standardized by the United States Bureau of Standards. Kerosene free from water, or benzine not lighter than 62° Baumé, shall be used in making this determination. **Apparatus.**

29. The flask shall be filled with either of these liquids to a point on the stem between zero and one cubic centimeter, and 64 g. of cement, of the same temperature as the liquid, shall be slowly introduced, taking care that the cement does not adhere to the inside of the flask above the liquid and to free the cement from air by rolling the flask in an inclined position. After all the cement is introduced, the level of the liquid will rise to some division of the graduated neck; the difference between readings is the volume displaced by 64 g. of the cement. **Method.**

The specific gravity shall then be obtained from the formula

$$\text{Specific gravity} = \frac{\text{Weight of cement (g.)}}{\text{Displaced volume (cc.)}}$$

30. The flask, during the operation, shall be kept immersed in water, in order to avoid variations in the temperature of the liquid in the flask, which shall not exceed 0°.5 C. The results of repeated tests should agree within 0.01.

31. The determination of specific gravity shall be made on the cement as received; if it falls below 3.10, a second determination shall be made after igniting the sample as described in Section 20.

IX. DETERMINATION OF FINENESS.

32. Wire cloth for standard sieves for cement shall be woven (not twilled) from brass, bronze, or other suitable wire, and mounted without distortion on frames not less than $1\frac{1}{2}$ in. below the top of the frame. The sieve frames shall be circular, approximately 8 in. in diameter, and may be provided with a pan and cover. **Apparatus**

33. A standard No. 200 sieve is one having nominally an 0.0029-in. opening and 200 wires per inch standardized by the

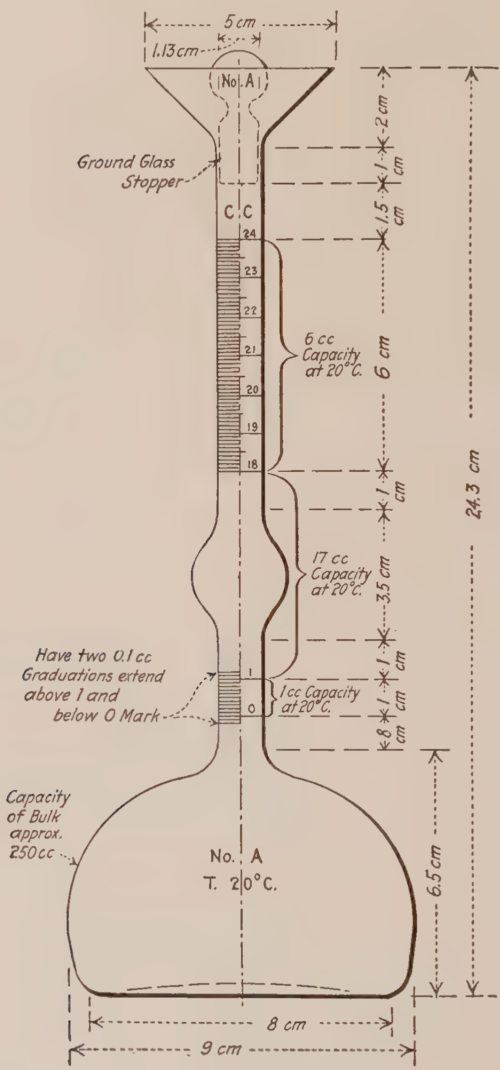


FIG. 1—Le Chatelier Apparatus.

U. S. Bureau of Standards, and conforming to the following requirements:

The No. 200 sieve should have 200 wires per inch, and the number of wires in any whole inch shall not be outside the limits of 192 to 208. No opening between adjacent parallel wires shall be more than 0.0050 in. in width. The diameter of the wire should be 0.0021 in. and the average diameter shall not be outside the limits 0.0019 to 0.0023 in. The value of the sieve as determined by sieving tests made in conformity with the standard specification for these tests on a standardized cement which gives a residue of 25 to 20 per cent on the No. 200 sieve, or on other similarly graded material, shall not show a variation of more than 1.5 per cent above or below the standards maintained at the Bureau of Standards.

34. The test shall be made with 50 g. of cement. The sieve shall be thoroughly clean and dry. The cement shall be placed on the No. 200 sieve, with pan and cover attached, if desired, and shall be held in one hand in a slightly inclined position so that the sample will be well distributed over the sieve, at the same time gently striking the side about 150 times per minute against the palm of the other hand on the up stroke. The sieve shall be turned every 25 strokes about one-sixth of a revolution in the same direction. The operation shall continue until not more than 0.05 g. passes through in one minute of continuous sieving. The fineness shall be determined from the weight of the residue on the sieve expressed as a percentage of the weight of the original sample. **Method.**

35. Mechanical sieving devices may be used, but the cement shall not be rejected if it meets the fineness requirement when tested by the hand method described in Section 34.

36. A permissible variation of 1 will be allowed, and all results in excess of the specified limit but within this permissible variation shall be reported as 22 per cent. **Permissible Variation.**

X. MIXING CEMENT PASTES AND MORTARS.

37. The quantity of dry material to be mixed at one time shall not exceed 1000 g. nor be less than 500 g. The proportions of cement or cement and sand shall be stated by weight in grams of the dry materials; the quantity of water shall be **Method.**

expressed in cubic centimeters (1 cc. of water = 1 g.). The dry materials shall be weighed, placed upon a non-absorbent surface, thoroughly mixed dry if sand is used, and a crater formed in the center, into which the proper percentage of clean water shall be poured; the material on the outer edge shall be turned into the crater by the aid of a trowel. After an interval of ½ minute for the absorption of the water the operation shall be completed by continuous, vigorous mixing, squeezing and kneading with the hands for at least one minute.[1] During the operation of mixing, the hands should be protected by rubber gloves.

38. The temperature of the room and the mixing water shall be maintained as nearly as practicable at 21° C. (70° F.).

XI. NORMAL CONSISTENCY.

Apparatus 39. The Vicat apparatus consists of a frame *A* (Fig. 2) bearing a movable rod *B*, weighing 300 g., one end *C* being 1 cm. in diameter for a distance of 6 cm., the other having a removable needle *D*, 1 mm. in diameter, 6 cm. long. The rod is reversible, and can be held in any desired position by a screw *E*, and has midway between the ends a mark *F* which moves under a scale (graduated to millimeters) attached to the frame *A*. The paste is held in a conical, hard-rubber ring *G*, 7 cm. in diameter at the base, 4 cm. high, resting on a glass plate *H* about 10 cm. square.

Method. 40. In making the determination, 500 g. of cement, with a measured quantity of water, shall be kneaded into a paste, as described in Section 37, and quickly formed into a ball with the hands, completing the operation by tossing it six times from one hand to the other, maintained about 6 in. apart; the ball resting in the palm of one hand shall be pressed into the larger end of the rubber ring held in the other hand, completely filling the ring with paste; the excess at the larger end shall then be removed by a single movement of the palm of the hand; the ring shall then be placed on its larger end on a glass plate and

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[1] In order to secure uniformity in the results of tests for the time of setting and tensile strength the manner of mixing above described should be carefully followed. At least one minute is necessary to obtain the desired plasticity which is not appreciably affected by continuing the mixing for several minutes. The exact time necessary is dependent upon the personal equation of the operator. The error in mixing should be on the side of over mixing.

the excess paste at the smaller end sliced off at the top of the ring by a single oblique stroke of a trowel held at a slight angle with the top of the ring. During these operations care shall be taken not to compress the paste. The paste confined in the ring, resting on the plate, shall be placed under the rod, the larger end of which shall be brought in contact with the surface

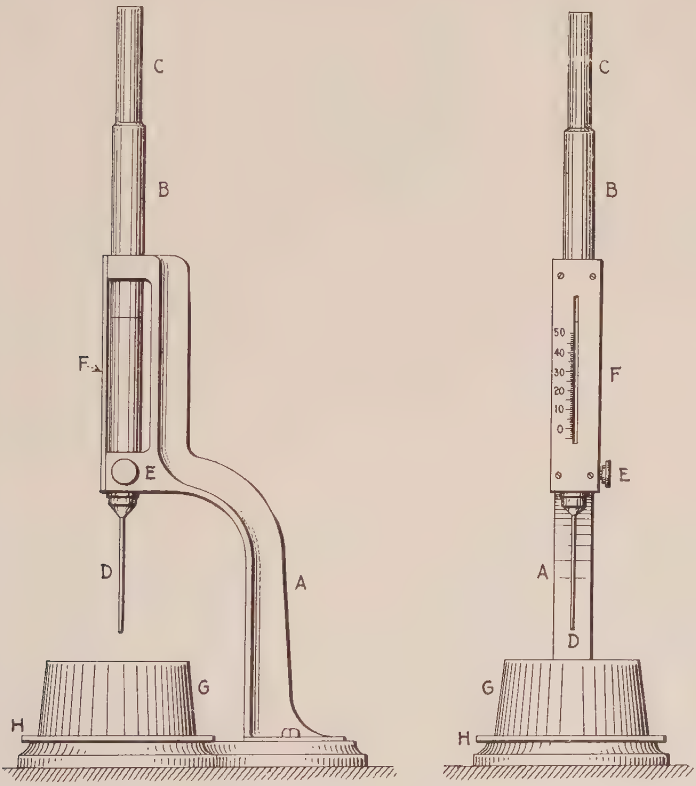


FIG. 2.—Vicat Apparatus.

of the paste; the scale shall be then read, and the rod quickly released. The paste shall be of normal consistency when the rod settles to a point 10 mm. below the original surface in ½ minute after being released. The apparatus shall be free from all vibrations during the test. Trial pastes shall be made with varying percentages of water until the normal consistency

is obtained. The amount of water required shall be expressed in percentage by weight of the dry cement.

41. The consistency of standard mortar shall depend on the amount of water required to produce a paste of normal consistency from the same sample of cement. Having determined the normal consistency of the sample, the consistency of standard mortar made from the same sample shall be as indicated in Table I, the values being in percentage of the combined dry weights of the cement and standard sand.

TABLE I.—PERCENTAGE OF WATER FOR STANDARD MORTARS.

Percentage of Water for Neat Cement Paste of Normal Consistency	Percentage of Water for One Cement, Three Standard Ottawa Sand.	Percentage of Water for Neat Cement Paste of Normal Consistency.	Percentage of Water for One Cement, Three Standard Ottawa Sand.
15	9.0	23	10.3
16	9.2	24	10.5
17	9.3	25	10.7
18	9.5	26	10.8
19	9.7	27	11.0
20	9.8	28	11.2
21	10.0	29	11.3
22	10.2	30	11.5

XII. DETERMINATION OF SOUNDNESS.[1]

Apparatus. 42. A steam apparatus, which can be maintained at a temperature between 98 and 100° C., or one similar to that shown in Fig. 3, is recommended. The capacity of this apparatus may be increased by using a rack for holding the pats in a vertical or inclined position.

Method. 43. A pat from cement paste of normal consistency about 3 in. in diameter, ½ in. thick at the center, and tapering to a thin edge, shall be made on clean glass plates about 4 in. square,

[1] Unsoundness is usually manifested by change in volume which causes distortion, cracking, checking or disintegration.

Pats improperly made or exposed to drying may develop what are known as shrinkage cracks within the first 24 hours and are not an indication of unsoundness. These conditions are illustrated in Fig. 4.

The failure of the pats to remain on the glass or the cracking of the glass to which the pats are attached does not necessarily indicate unsoundness.

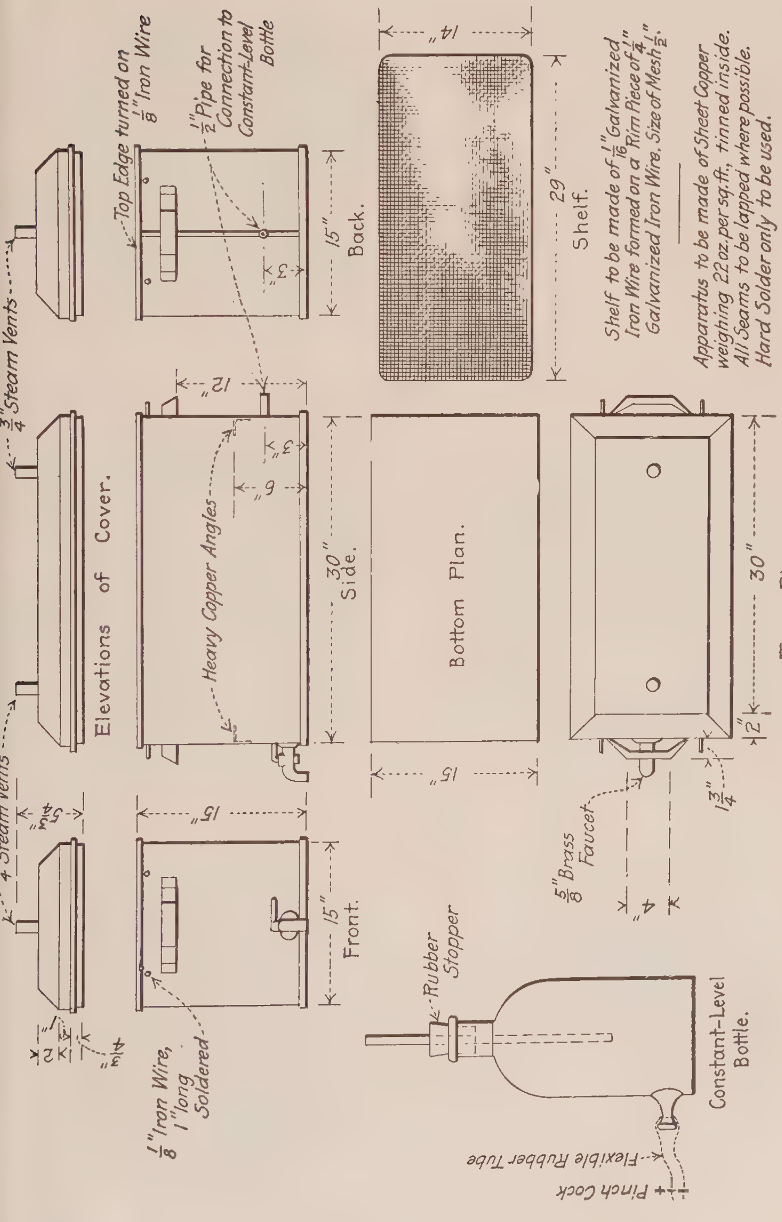
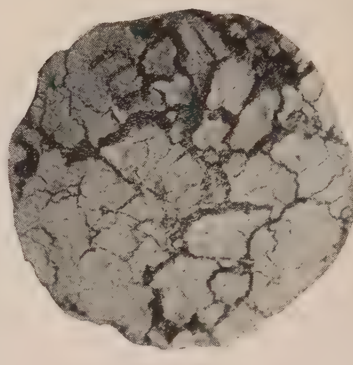
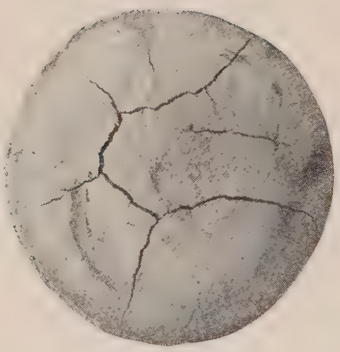


FIG. 3.—Apparatus for Making Soundness Test of Cement.

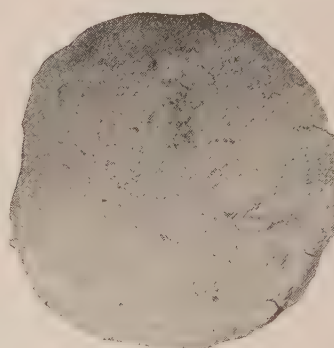
Distortion.



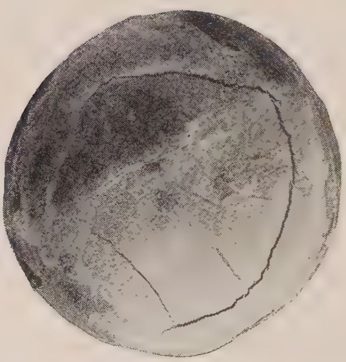
Disintegration.



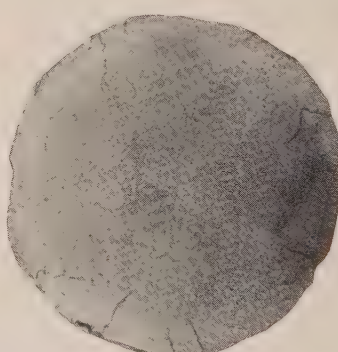
Shrinkage.



Checking.



Shrinkage.



Cracking.

and stored in moist air for 24 hours. In molding the pat, the cement paste shall first be flattened on the glass and the pat then formed by drawing the trowel from the outer edge toward the center.

44. The pat shall then be placed in an atmosphere of steam at a temperature between 98 and 100° C. upon a suitable support 1 in. above boiling water for 5 hours.

45. Should the pat leave the plate, distortion may be detected best with a straight edge applied to the surface which was in contact with the plate.

XIII. DETERMINATION OF TIME OF SETTING.

46. The following are alternate methods, either of which may be used as ordered:

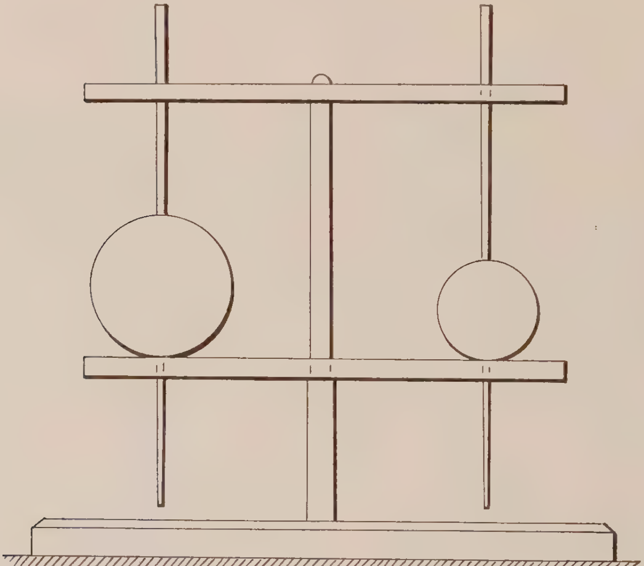
47. The time of setting shall be determined with the Vicat apparatus described in Section 39. (See Fig. 2.) **Vicat Apparatus.**

48. A paste of normal consistency shall be molded in the hard-rubber ring *G* as described in Section 40, and placed under the rod *B*, the smaller end of which shall then be carefully brought in contact with the surface of the paste, and the rod quickly released. The initial set shall be said to have occurred when the needle ceases to pass a point 5 mm. above the glass plate in ½ minute after being released; and the final set, when the needle does not sink visibly into the paste. The test pieces shall be kept in moist air during the test. This may be accomplished by placing them on a rack over water contained in a pan and covered by a damp cloth, kept from contact with them by means of a wire screen; or they may be stored in a moist closet. Care shall be taken to keep the needle clean, as the collection of cement on the sides of the needle retards the penetration, while cement on the point may increase the penetration. The time of setting is affected not only by the percentage and temperature of the water used and the amount of kneading the paste receives, but by the temperature and humidity of the air, and its determination is therefore only approximate. **Vicat Method.**

49. The time of setting shall be determined by the Gillmore needles. The Gillmore needles should preferably be mounted as shown in Fig. 5 (*b*). **Gillmore Needles.**

Gillmore Method 50. The time of setting shall be determined as follows: A pat of neat cement paste about 3 in. in diameter and $\frac{1}{2}$ in. in thickness with a flat top (Fig. 5 (*a*)), mixed to a normal consistency, shall be kept in moist air at a temperature maintained as nearly as practicable at 21° C. (70° F.). The cement shall be considered to have acquired its initial set when the pat will bear,

(*a*) Pat with Top Surface Flattened for Determining Time of Setting by Gillmore Method.



(*b*) Gillmore Needles.

FIG. 5.

without appreciable indentation, the Gillmore needle $\frac{1}{12}$ in. in diameter, loaded to weigh $\frac{1}{4}$ lb. The final set has been acquired when the pat will bear without appreciable indentation, the Gillmore needle $\frac{1}{24}$ in. in diameter, loaded to weigh 1 lb. In making the test, the needles shall be held in a vertical position, and applied lightly to the surface of the pat.

XIV. TENSION TESTS.

51. The form of test piece shown in Fig. 6 shall be used. The molds shall be made of non-corroding metal and have sufficient material in the sides to prevent spreading during molding. Gang molds when used shall be of the type shown in Fig. 7. Molds shall be wiped with an oily cloth before using. **Form of Test Piece.**

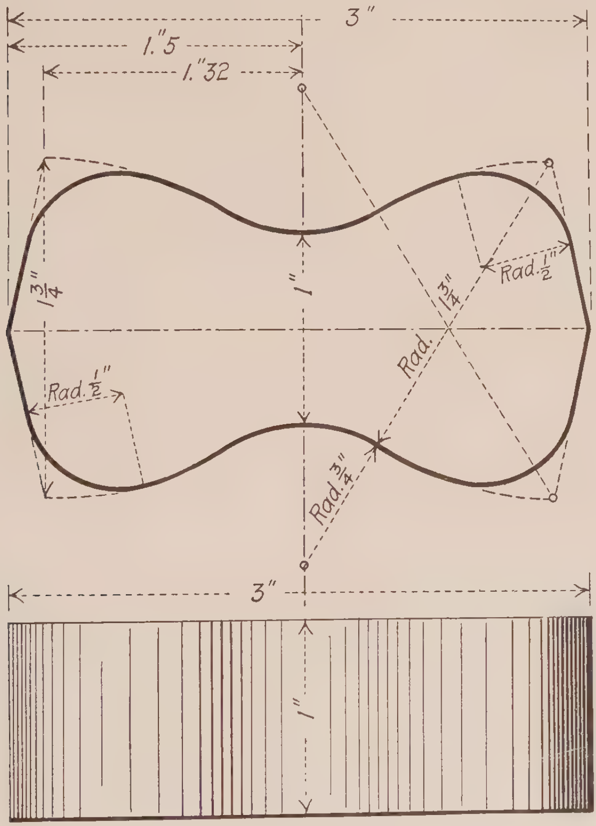


FIG. 6.—Details for Briquette.

52. The sand to be used shall be natural sand from Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve. This sand may be obtained from the Ottawa Silica Co., at a cost of two cents per pound, f. o. b. cars, Ottawa, Ill. **Standard Sand.**

53. This sand, having passed the No. 20 sieve, shall be considered standard when not more than 5 g. pass the No. 30 sieve after one minute continuous sieving of a 500-g. sample.

54. The sieves shall conform to the following specifications:

The No. 20 sieve shall have between 19.5 and 20.5 wires per whole inch of the warp wires and between 19 and 21 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0165 in. and the average diameter shall not be outside the limits of 0.0160 and 0.0170 in.

The No. 30 sieve shall have between 29.5 and 30.5 wires per whole inch of the warp wires and between 28.5 and 31.5 wires per whole inch of the shoot wires. The diameter of the wire should be 0.0110 in. and the average diameter shall not be outside the limits 0.0105 to 0.0115 in.

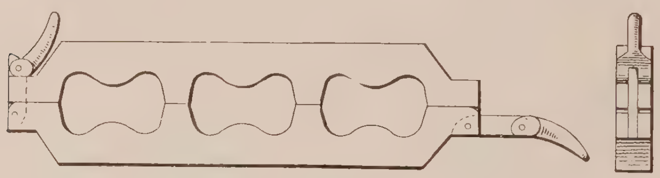


FIG. 7.—Gang Mold.

Molding. 55. Immediately after mixing, the standard mortar shall be placed in the molds, pressed in firmly with the thumbs and smoothed off with a trowel without ramming. Additional mortar shall be heaped above the mold and smoothed off with a trowel; the trowel shall be drawn over the mold in such a manner as to exert a moderate pressure on the material. The mold shall then be turned over and the operation of heaping, thumbing and smoothing off repeated.

Testing. 56. Tests shall be made with any standard machine. The briquettes shall be tested as soon as they are removed from the water. The bearing surfaces of the clips and briquettes shall be free from grains of sand or dirt. The briquettes shall be carefully centered and the load applied continuously at the rate of 600 lb. per minute.

57. Testing machines should be frequently calibrated in order to determine their accuracy.

58. Briquettes that are manifestly faulty, or which give strengths differing more than 15 per cent from the average value of all test pieces made from the same sample and broken at the same period, shall not be considered in determining the tensile strength. **Faulty Briquettes.**

XV. STORAGE OF TEST PIECES.

59. The moist closet may consist of a soapstone, slate or concrete box, or a wooden box lined with metal. If a wooden box is used, the interior should be covered with felt or broad wicking kept wet. The bottom of the moist closet should be covered with water. The interior of the closet should be provided with non-absorbent shelves on which to place the test pieces, the shelves being so arranged that they may be withdrawn readily. **Apparatus.**

60. Unless otherwise specified all test pieces, immediately after molding, shall be placed in the moist closet for from 20 to 24 hours. **Methods.**

61. The briquettes shall be kept in molds on glass plates in the moist closet for at least 20 hours. After 24 hours in moist air the briquettes shall be immersed in clean water in storage tanks of non-corroding material.

62. The air and water shall be maintained as nearly as practicable at a temperature of 21° C. (70° F.).

AMERICAN SOCIETY FOR TESTING MATERIALS

PHILADELPHIA, PA., U. S. A.

AFFILIATED WITH THE

INTERNATIONAL ASSOCIATION FOR TESTING MATERIALS.

STANDARD SPECIFICATIONS

FOR

DRAIN TILE.

Serial Designation: C 4 – 16.

The specifications for this material are issued under the fixed designation C 4; the final number indicates the year of original issue, or in the case of revision, the year of last revision.

ADOPTED, 1914; REVISED, 1916.

Classes.

1. (*a*) These specifications cover three classes of drain tile, namely, Farm Drain Tile, Standard Drain Tile, and Extra-Quality Drain Tile.

(*b*) The purposes for which these classes are intended to be suitable are as follows:

Farm Drain Tile, for ordinary private drainage work on farms, for moderate sizes and depths;

Standard Drain Tile, for ordinary district land drainage at moderate depths;

Extra-Quality Drain Tile, for district land drainage, for considerable depths and where an extra quality is desired.

Basis of Purchase.

2. The purchaser shall specify the class or classes of tile to be supplied, whether Farm Drain Tile, Standard Drain Tile, or Extra-Quality Drain Tile. Standard Drain Tile shall be supplied where no other advance selection is stated.

Basis of Acceptance.

3. (*a*) The acceptability of drain tile shall be determined (1) by the results of the chemical and physical tests hereinafter

specified, and (2) by visual inspection, to determine whether the tiles comply with the specifications as to dimensions, shape, and freedom from visible external and internal defects.

(*b*) The acceptance of drain tile as satisfactorily meeting one of these two general requirements shall not be construed as in any way waiving the other.

I. MATERIALS AND MANUFACTURE.

4. (*a*) These specifications shall apply to drain tile made of shale, fire clays or surface clays and to drain tile made of concrete. **Materials.**

(*b*) By shale is meant a stratified clay, usually red-burning, more or less indurated by heat or pressure, with well-marked cleavage, laid down prior to the present geological epoch.

(*c*) By fire clay is meant a stratified clay, usually buff-burning, usually less indurated than shales, with poorly marked cleavage, laid down prior to the present geological epoch.

(*d*) By surface clay is meant an unstratified, unconsolidated plastic glacial or alluvial clay, laid down by the glacial ice sheet, or on the flood plains of rivers, during the present geological epoch.

(*e*) By concrete is meant a suitable mixture of Portland cement, mineral aggregates and water, hardened by hydraulic chemical reaction.

(*f*) If the purchaser desires to exclude any of these materials he shall so specify in advance, All materials used shall be first-class of their kind and suitable for the purpose.

5. The method of manufacture shall be such as to insure excellence of product and uniformity in quality. **Manufacture.**

II. CHEMICAL TESTS AND REQUIREMENTS.

6. The purchaser may prescribe in advance special chemical requirements in cases where drainage waters have marked acid or alkaline character, or are of abnormally high temperature, and may use chemical analysis of the tile to ascertain whether these special requirements are met. Without a special agreement in advance, no drain tile shall be rejected by reason of its composition as determined by ultimate chemical analysis. **Chemical Tests and Requirements.**

The presence in drain tile of visible grains or masses of caustic lime, iron pyrites, or any other minerals which are

known to cause slaking or disintegration of the tile, shall be construed as a valid ground for rejection, unless satisfactory proof be submitted that the tiles are permanent and durable, and that the objectionable minerals are not present in quantity or condition to work damage.

III. PHYSICAL TESTS.

Physical Tests.

7. The physical tests of drain tile shall include (*A*) Strength Tests and (*B*) Absorption Tests; and may include (*C*) Freezing and Thawing Tests, when specified by the purchaser in advance, or when called for by the manufacturer or other seller as provided in Sections 34, 35, 47 and 52.

Selection of Specimens of Tile.

8. The specimens of tile shall all be selected at the factory or at the shipping destination, or at the trench, at the option of the purchaser. The selection shall be made by a competent inspector employed by the purchaser. The inspector shall divide the tile into sub-classes if lack of uniformity in any important particular warrants such division, and shall select enough representative specimens of tile from each sub-class for a complete set of standard physical tests.

Number and Cost of Specimens of Tile.

9. A standard physical test shall comprise tests of five individual tiles. Specimens of tile may be selected by the inspector in such number as he judges necessary to determine fairly the quality of all the tile. The manufacturer or other seller shall furnish specimens of tile without separate charge up to 0.5 per cent of the whole number of tile, and the purchaser shall pay for all in excess of that percentage at the same rate as for other tile.

(*A*) *Strength Tests of Drain Tile.*

Specimens of Tile.

10. The specimens of tile shall be unbroken, full-size tile.

Moisture Condition of Specimens of Tile

11. The walls of the tile shall, at the time of testing, be as thoroughly wet as will result from completely covering with hay, cloth, or similar absorbent material, and keeping the covering wet for not less than 12 hours.

Temperature Condition of Specimens of Tile.

12. No specimen of tile shall be exposed to water or air temperatures lower than 40° F. from the beginning of wetting until tested. Frozen tile shall be completely thawed before the wetting begins.

13. Each specimen of tile shall be weighed on reliable scales just prior to testing, and the weights shall be reported. **Weighing.**

14. Any machine or hand method which will apply the load continuously, or in increments not exceeding 5 per cent of the estimated total breaking load, may be used in making the test. The tile shall not be allowed to stand under load longer than is required for observing and recording the loads. All solid parts of the bearing frames and bearing blocks shall be so rigid that the distribution of the load will not be affected appreciably by the deformation of any part. All bearings and the specimens of tile shall be so accurately centered as to secure a symmetrical distribution of the loading on each side of the center of the tile in every direction. **Application of Load.**

15. The purchaser shall choose (1) sand bearings, (2) hydraulic bearings, or (3) three-point bearings, for use in making strength tests of drain tile. (See Sections 18, 19, and 20). **Choice of Bearings.**

16. The test results shall be calculated and reported, in pounds per linear foot of tile, in terms of the "Ordinary Supporting Strength."[1] **Calculation and Reporting of Test Results.**

The ordinary supporting strength shall be calculated by multiplying the test breaking loads by the following factors: For sand bearings, 1.00; for hydraulic bearings, 1.25; for three-point bearings, 1.50.

The results of the strength tests shall be reported separately for each of the five individual specimens of tile constituting a standard test, together with the average.

17. The modulus of rupture may or may not be calculated and reported, at the option of the purchaser. When reported it shall be calculated by the equations[2] **Modulus of Rupture.**

$$M=0.20\,r\,\frac{W}{12}\qquad\qquad (1)$$

$$f=\frac{6M}{t^2}\qquad\qquad (2)$$

---

[1] The "ordinary supporting strength," when calculated as specified in Section 16, is approximately equal to the actual supporting strength of a tile when laid in a ditch by the "ordinary" method. See note under Table II.

[2] The coefficient of 0.20 in equation (1) approximates the value found by theoretical analysis and also that determined by extended tests.

where M = maximum bending moment in wall in pound-inches per inch of length, r = radius of middle line of tile wall in inches, W = ordinary supporting strength, calculated as prescribed in Section 16, in pounds per linear foot of tile, f = modulus of rupture in pounds per square inch, and t = thickness of tile wall in inches.

Five-eighths of the weight of the tile per linear foot for sand bearings, or three-fourths for hydraulic or three-point bearings, shall be added to W in computing the maximum bending moment M, when such addition exceeds 5 per cent of W. The value of t used shall be the average thickness of the wall at the top of the tile or that at the bottom, selecting the lesser of the two.

Sand Bearings. 18. (See Fig. 1.)—When sand bearings are used, the ends of each specimen of tile shall be accurately marked in quarters of the circumference prior to the test. Specimens shall be carefully bedded, above and below, in sand, for one-fourth the circumference of the tile measured on the middle line of the wall. The depth of bedding above and below the tile at the thinnest points shall be one-half the radius of the middle line of the wall.

The sand used shall be clean, and shall be such as will pass a No. 4 screen.

The top bearing frame shall not be allowed to come in contact with the tile nor with the top bearing plate. The upper surface of the sand in the top bearing shall be struck level with a straight edge, and shall be covered with a rigid top bearing plate, with lower surface a true plane, made of heavy timbers or other rigid material, capable of distributing the test load uniformly without appreciable bending. The test load shall be applied at the exact center of this top bearing plate, in such a manner as to permit free motion of the plate in all directions. For this purpose a spherical bearing is preferred, but two rollers at right angles may be used. The test may be made without the use of a testing machine, by piling weights directly on a platform resting on the top bearing plate, provided, however, that the weight shall be piled symmetrically about a vertical line through the center of the tile, and that the platform shall not be allowed to touch the top bearing frame.

The frames of the top and bottom bearings shall be made

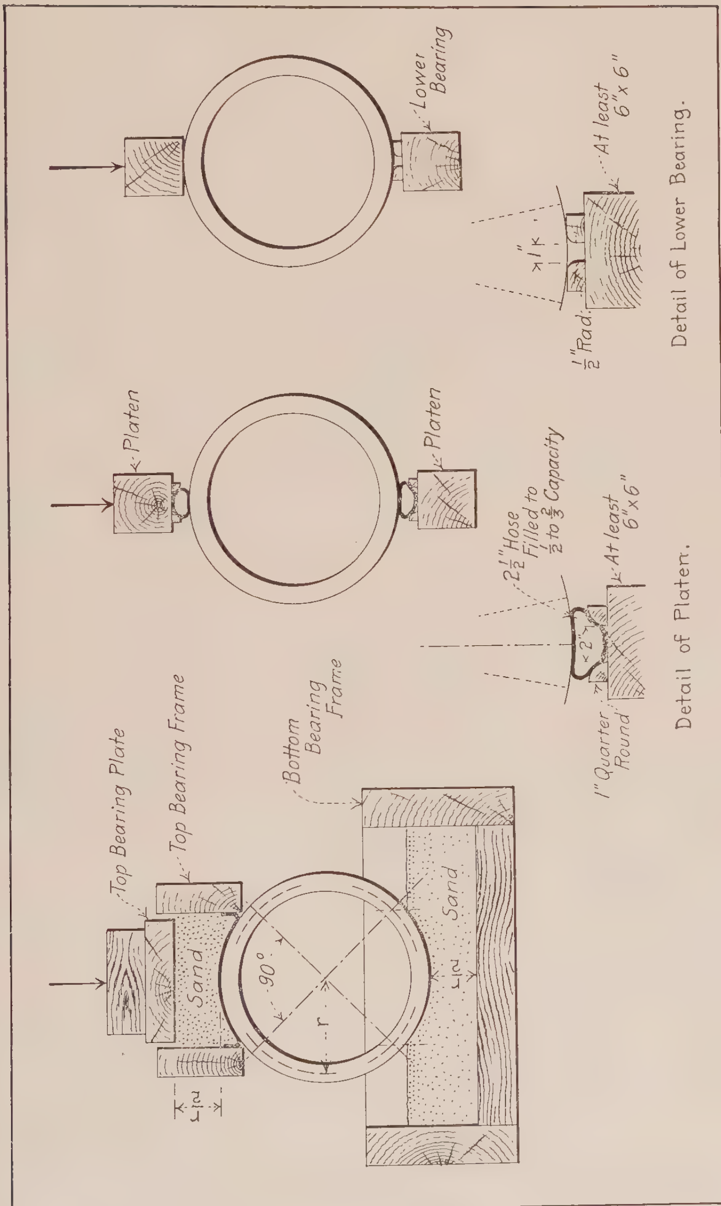


FIG. 1.—Sand Bearings

FIG. 2.—Hydraulic Bearings.

FIG. 3.—Three-Point Bearings.

of timbers so heavy as to avoid appreciable bending by the side pressure of the sand. The interior surfaces of the frames shall be dressed. No frame shall come in contact with the tile during the test. A strip of cloth may, if desired, be attached to the inside of the upper frame on each side, along the lower edge, to prevent the escape of sand between the frame and the tile.

Hydraulic Bearings.

19. (See Fig. 2.)—When hydraulic bearings are used, the ends of each specimen of tile shall be accurately marked in halves of the circumference prior to the test.

An hydraulic bearing shall be composed of a wooden platen, to which is attached, as hereinafter described, a section of rubber hose. The hose shall lie against the tile, and the pressure shall be applied to the hose through the platen.

The platen shall be built of strong wood, and shall be not less than 6 by 6 in. in section, and its least length shall be the length of the tile plus 8 in. One-inch quarter rounds, with their convex surfaces facing and 2 in. apart in the clear, shall be firmly attached to the bearing side. The straight portion of this face shall extend at least the length of the tile, and the platen beyond this length may be cut to the arc of a circle.

Between the quarter rounds shall be laid a piece of $2\frac{1}{2}$-in. hose which shall be closed in a water-tight manner at each end by clamps. The hose shall contain a volume of water not less than one-half nor more than two-thirds its capacity, when completely distended. This hose may be attached to the platen at either end in any satisfactory manner which will not induce wrinkling when under test pressure.

The test load shall be applied at the middle of the top bearing, in such a way as to leave the bearing free to move in the vertical plane of the axis of the tile.

It is recommended that stops be screwed to the platen, symmetrical with the point of application of the load, and at a distance apart not greater than the length of the tile plus $\frac{1}{2}$ in. This will help center the load coming upon the tile.

Three-point Bearings.

20. (See Fig. 3.)—When three-point bearings are used, the ends of each specimen of tile shall be accurately marked in halves of the circumference prior to the test.

The lower bearings shall consist of two wooden strips with

vertical sides, each strip having its interior top corner rounded to a radius of approximately $\frac{1}{2}$ in. They shall be straight, and shall be securely fastened to a rigid block with their interior vertical sides 1 in. apart.

The upper bearing shall be a wooden block, straight and true from end to end.

The test load shall be applied through the upper bearing block in such a way as to leave the bearing free to move in a vertical plane passing midway between the lower bearings.

In testing a tile which is "out of straight," the lines of the bearings chosen shall be from those which appear to give most favorable conditions for fair bearings.

(*B*) *Absorption Tests of Drain Tile.*

21. Not less than three separate test specimens from each of five separate tiles shall be taken as a "standard sample" for the absorption test. Of the three specimens from each tile, one shall be taken from one end, another from the opposite end, and the third shall be taken from the middle portion of the tile. Each specimen shall be of from 12 to 20 sq. in. in area, measured upon the exterior or convex side, and shall be as nearly square as the nature of the material will readily permit. The specimens shall be obtained by breaking the tile, and shall be apparently sound, solid pieces of the wall of the tile, and shall not show cracks or fissures or shattered edges due to the shock of breaking or cutting. The specimens may be obtained from the broken pieces of the tiles used in the strength test, if the restrictions as to the size and location of the specimens can be duly observed. The specimens shall be so marked as to permit the identity of each one to be ascertained at any stage of the test. **Test Specimens**

22. Preparatory to the absorption test, all specimens shall be first weighed and then dried in a drier or oven, at a temperature of not less than 110° C. (230° F.) for not less than three hours. After removal from the drier, the specimens shall be allowed to cool to a temperature of 20 to 25° C. (68 to 77° F.) and reweighed. If the specimens were apparently dry when taken, and the second weight closely checks the first, the specimens shall be considered dry. If the specimens were known to **Drying Test Specimens.**

be wet when taken, they shall be placed in the drier for a further drying treatment of two hours, and reweighed. If the third weight checks the second, the specimens shall be considered dry. In case of any doubt, the specimens must be redried for two-hour periods until check weights are obtained.

Weighing and Reweighing.

23. The balance used shall be sensitive to 0.5 g. when loaded with 1 kg., and weighings shall be read at least to the nearest gram. Where other than metric weights are used, the same order of accuracy must be obtained.

In reweighing after immersion, the specimens shall be removed from the water, not allowed to drain for more than one minute, the superficial water removed by towel or blotting paper, and the specimens at once put upon the balance.

Immersion of Test Specimens.

24. Specimens after weighing shall be placed in a suitable woven-wire receptacle, packed tightly enough to prevent jostling, covered with distilled water or rainwater, raised to the boiling point and boiled for five hours, and then cooled in water to a final temperature of 10 to 15° C. (50 to 59° F.).

Calculation and Reporting of Results.

25. The test results shall be calculated as percentages of the initial dry weight, carried to the nearest first decimal place. The results shall be reported separately for each individual specimen, together with the mean of the fifteen or more specimens comprising the standard sample, the maximum and the minimum single observations entering into the mean, and the variation between the maximum and the minimum of the three specimens of each tile represented in the the standard sample.

(C) *Freezing and Thawing Tests of Drain Tile.*

Test Specimens.

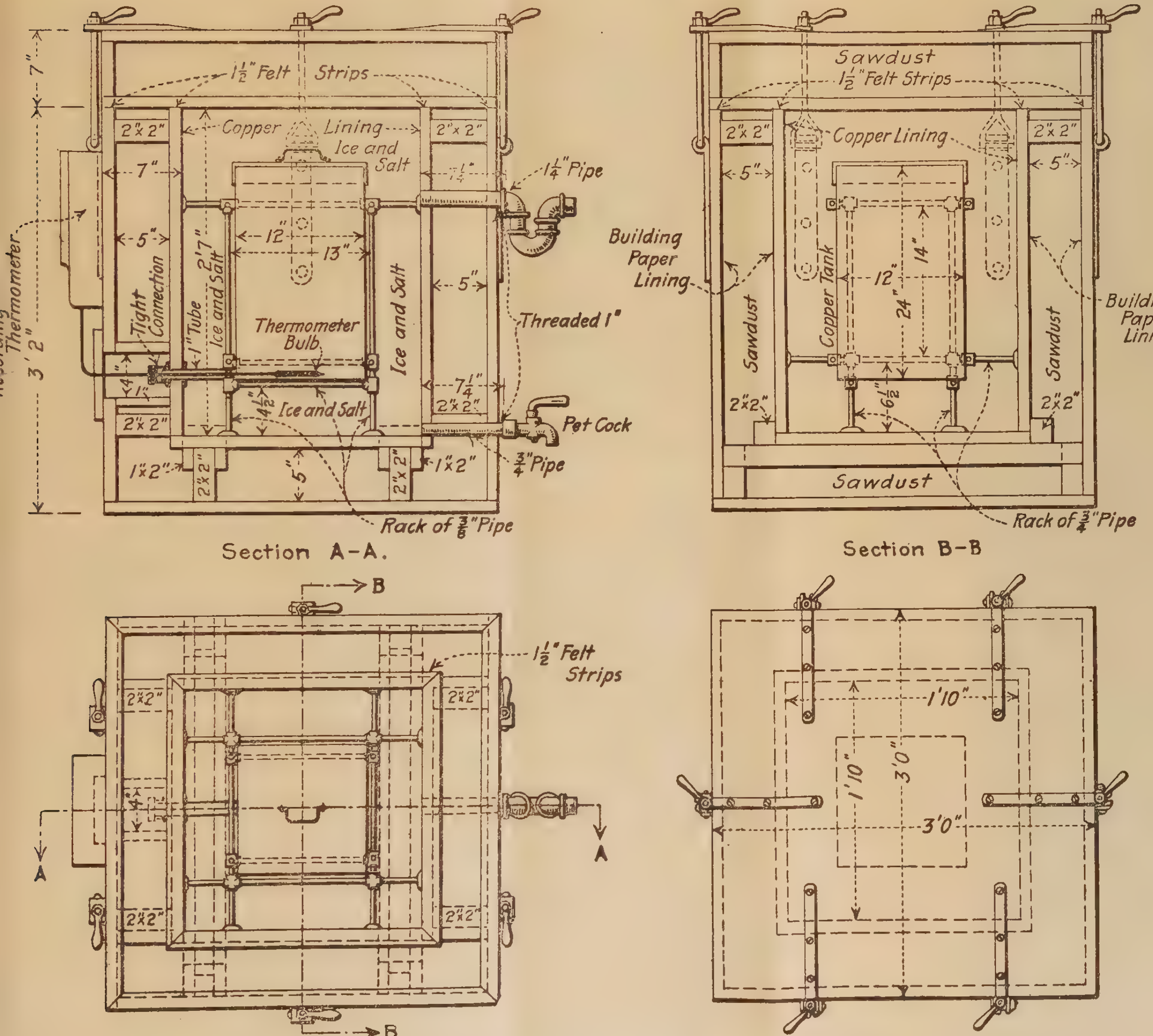
26. The test specimens employed in making the absorption test shall preferably be used for the freezing and thawing test. In the event that the same specimens are not available, another set selected as specified in Section 21 shall be taken.

Drying Test Specimens.

27. In the event that new specimens for the freezing and thawing test must be prepared, they shall be dried as specified in Section 22.

Weighing and Reweighing.

28. The same scales and weights as are specified in Section 23 for the absorption test or others of equivalent sensitiveness and accuracy shall be employed for the weighings required in



Section A–A.

Section B–B

Plan View, Top Removed.

Plan View, Top in Place

Note: *Box to be Constructed of Seasoned White Pine, free from Defects, or other Suitable Timber, 1" x 6", unless otherwise Specified.*

SUGGESTED PLANS
FOR
FREEZING BOX
USING SALT AND ICE TO FREEZE

the freezing and thawing test. The same procedure in weighings and reweighing as specified in Section 23 shall be used.

Immersion of Test Specimens.

29. In the event that new specimens for the freezing and thawing test must be prepared, they shall be immersed and boiled and cooled in water as specified in Section 24.

Freezing and Thawing.

30. When the specimens (either from the absorption test or from a specially prepared series) have been weighed after saturation with water, they shall be returned to the water, and kept immersed till the freezing test is begun. For freezing, they shall be placed with their concave faces upward in watertight metal trays, suitably mounted in a rigid metal crate,[1] and immersed in ice water until the specimens have attained substantially the temperature of the water, after which the water shall be drawn down to a depth of $\frac{1}{2}$ in. in each tray. The crate shall then be lifted as a whole, without disturbing the specimens, and placed in the freezing apparatus.

Freezing shall be performed in a quiet atmosphere, free from perceptible natural or artificial currents. If artificial freezing apparatus is employed,[2] the apparatus shall have sufficient heat-absorbent capacity to enable the temperature of the freezing chamber to be brought to $-10°$ C. ($+14°$ F.) or below, within thirty minutes after the introduction of the specimens. The temperature in the freezing apparatus shall not fall lower than $-20°$ C. ($-4°$ F.). The freezing shall be continued until the water in the trays is frozen solid. Exposure to freezing conditions in excess of this requirement shall be considered as without significance.

At the conclusion of freezing under the specified conditions, the crate of specimens shall be withdrawn and at once immersed in water at a temperature of 85 to 100° C. (185 to 212° F.) in a special receptacle of proper size. Heating shall be continued so that the water will regain the required temperature as soon as practicable after the specimens are immersed. A temperature of 85 to 100° C. (185 to 212° F.) shall then be maintained for not less than 15 minutes. At the conclusion of

[1] Fig. 4 shows a crate and trays suitable for use in the box for artificial freezing illustrated in Plate I.

[2] Artificial freezing will generally be necessary. It may be conducted in a commercial zero (F.) refrigerating room, or in an artificial freezing box similar to the one shown in Plate I, in which zero (F.) temperatures can readily be produced by the use of salt and ice.

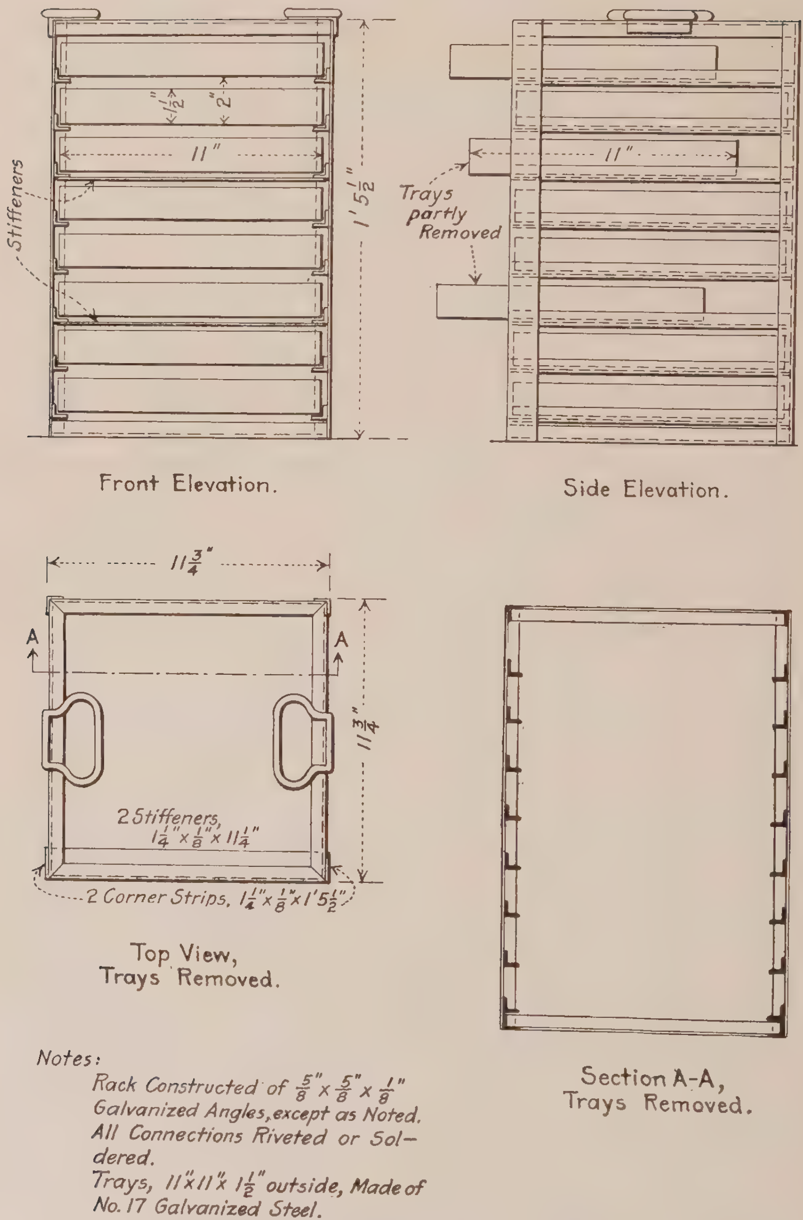


FIG. 4.—Suggested Plans for Freezing Crate and Trays.

the thawing treatment, the crate of specimens shall be cooled down rapidly in water to 10 to 15° C. (50 to 59° F.) and then inspected. The condition of each sample after each thawing shall be noted in the records.

Method of Determining Failure in Freezing and Thawing Tests

31. Failure under the freezing and thawing treatment shall be considered to be reached when:

(*a*) The specimens show superficial disintegration or spalling with loss of weight of more than 5 per cent of the initial dry weight; or,

(*b*) The specimens are badly cracked in other than lamination planes; or,

(*c*) The specimens show evident serious loss of structural strength.

IV. PHYSICAL TEST REQUIREMENTS.

Physical Test Requirements.

32. The physical test requirements for the different classes of drain tile shall be as given in Table I.

Absorption Requirements for Drain Tile made of Mixed Clays.

33. Drain tile made of mixtures of surface clays with other clays shall conform to the absorption requirements for surface-clay tile in Table I, when the proportion of surface clay is 75 per cent or more, and to the requirements for shale and fire-clay tile for all other proportions.

Appeal from Absorption Test to Freezing and Thawing Test.

34. In the event that a standard sample (Section 21) of tile fails to meet the requirements of the absorption test, the manufacturer or other seller may demand recourse to the freezing and thawing test, to be made at his expense. In such recourse, the number of tiles tested shall be four times the number represented by the standard sample (Section 21). If the material passes the freezing and thawing test satisfactorily, it shall not be rejected on account of its failure to meet the absorption requirements specified in Table I, but the average percentage of absorption of the specimens used in the freezing and thawing test shall be adopted as the maximum allowable mean absorption for the contract in question.

Limits of Fluctuation of Individual Test Specimens in Physical Tests; Culling and Retesting when Limits Exceeded.

35. In the strength tests, individual tiles of a standard test whose mean strength is satisfactory may fall 25 per cent below the requirement for the average without causing rejection. In the absorption test, the absorption of individual tiles of a standard sample (Section 21), which gives a satisfactory mean

absorption percentage, may exceed the average by 25 per cent without causing rejection. In the freezing and thawing test, at least 95 per cent of all the tiles tested shall meet the requirement.

In the event of the failure of a standard sample (Sections 9, 21 and 26) to meet the above requirements, the manufacturer

TABLE I.—PHYSICAL-TEST REQUIREMENTS FOR DIFFERENT CLASSES OF DRAIN TILE.

Internal Diameter of Tile, in.	Farm Drain Tile.				Standard Drain Tile.				Extra-Quality Drain Tile.			
	Minimum Average Ordinary Supporting Strength, lb. per linear ft.	Maximum Average Absorption by Standard Boiling Test, per cent.			Minimum Average Ordinary Supporting Strength, lb. per linear ft.	Maximum Average Absorption by Standard Boiling Test, per cent.			Minimum Average Ordinary Supporting Strength, lb. per linear ft.	Maximum Average Absorption by Standard Boiling Test, per cent.		
		Shale and Fire-Clay Tile.	Surface-Clay Tile.	Concrete Tile.		Shale and Fire-Clay Tile.	Surface-Clay Tile.	Concrete Tile.		Shale and Fire-Clay Tile.	Surface-Clay Tile.	Concrete Tile.
4	800	11	14	12	1200	9	13	11	1600	7	11	10
6	800	11	14	12	1200	9	13	11	1600	7	11	10
8	800	11	14	12	1200	9	13	11	1600	7	11	10
10	800	11	14	12	1200	9	13	11	1600	7	11	10
12	800	11	14	12	1200	9	13	11	1600	7	11	10
14	900	11	14	12	1200	9	13	11	1600	7	11	10
16	1000	11	14	12	1300	9	13	11	1600	7	11	10
18	(Not Permitted)	(Not Permitted)	(Not Permitted)	(Not Permitted)	1400	9	13	11	1800	7	11	10
20					1500	9	13	11	2000	7	11	10
22					1600	9	13	11	2200	7	11	10
24					1700	9	13	11	2400	7	11	10
26					1800	9	13	11	2600	7	11	10
28					1900	9	13	11	2800	7	11	10
30					2000	9	13	11	3000	7	11	10
32					2100	9	13	11	3200	7	11	10
34					2200	9	13	11	3400	7	11	10
36					2300	9	13	11	3600	7	11	10
38					2400	9	13	11	3800	7	11	10
40					2500	9	13	11	4000	7	11	10
42					2600	9	13	11	4200	7	11	10

NOTE.—When the freezing and thawing test is specified or demanded, as provided in Section 7, the number of freezings and thawings to be endured shall be as follows: For farm drain tile, 8; for standard drain tile, 12; for extra-quality drain tile, 16.

or other seller may thoroughly cull the material and submit a portion of retest at his own expense, and for such retest the number of tiles per sample shall be 10 for the strength and absorption tests and 20 for the freezing and thawing test. In the event of the failure of the material after culling to pass the requirements, it shall be rejected without further test.

TABLE II.—STANDARD ORDINARY SUPPORTING STRENGTHS OF DRAIN TILE FOR ORDINARY SAND AND FOR THOROUGHLY WET CLAY DITCH FILLING MATERIALS.

STRENGTHS IN POUNDS PER LINEAR FOOT.

Height of Fill above Top of Tile, ft.	Breadth of Ditch at Top of Tile.																			
	1 ft.				2 ft				3 ft.				4 ft.				5 ft.			
	Method of Laying Pipe.				Method of Laying Pipe.				Method of Laying Pipe.				Method of Laying Pipe.				Method of Laying Pipe.			
	Ordinary.		First Class.		Ordinary.		First Class.		Ordinary.		First Class.		Ordinary.		First Class.		Ordinary.		First Class.	
	Ditch Filling Material.		Ditch Filling Material.		Ditch Filling Material.		Ditch Filling Material.		Ditch Filling Material.		Ditch Filling Material.		Ditch Filling Material.		Ditch Filling Material.		Ditch Filling Material.		Ditch Filling Material.	
	Sand.	Clay.	Sand.	Clay.	Sand.	Clay.	Sand.	Clay.	Sand.	Clay.	Sand.	Clay.	Sand.	Clay.	Sand.	Clay.	Sand.	Clay.	Sand.	Clay.
2	265	280	220	235	615	635	510	530	970	990	810	830	1 330	1 350	1 110	1 130	1 690	1 710	1 410	1 430
4	400	450	335	375	1 050	1 120	880	935	1 750	1 820	1 460	1 520	2 450	2 540	2 040	2 110	3 160	3 250	2 640	2 710
6	470	545	390	455	1 370	1 500	1 140	1 250	2 370	2 530	1 980	2 110	3 410	3 580	2 840	2 980	4 460	4 640	3 720	3 870
8	505	605	420	505	1 600	1 790	1 330	1 490	2 870	3 110	2 390	2 590	4 220	4 490	3 510	3 740	5 590	5 890	4 660	4 910
10	525	640	440	535	1 760	2 010	1 470	1 680	3 270	3 610	2 730	3 010	4 900	5 290	4 080	4 410	6 590	7 020	5 490	5 850
12	535	660	445	550	1 880	2 190	1 570	1 820	3 600	4 030	3 000	3 355	5 480	6 000	4 570	5 000	7 460	8 030	6 220	6 690
14	540	675	450	560	1 960	2 320	1 640	1 930	3 850	4 380	3 210	3 650	5 980	6 620	4 980	5 520	8 230	8 950	6 850	7 460
16	545	680	455	565	2 030	2 420	1 690	2 020	4 060	4 670	3 390	3 890	6 400	7 160	5 330	5 970	8 890	9 770	7 410	8 150
18	545	685	455	570	2 070	2 500	1 720	2 090	4 230	4 920	3 530	4 100	6 750	7 630	5 630	6 360	9 480	10 520	7 900	8 770
20	545	690	455	575	2 100	2 560	1 750	2 140	4 370	5 130	3 640	4 280	7 050	8 060	5 880	6 720	9 990	11 200	8 330	9 330
22	545	690	455	575	2 120	2 610	1 770	2 180	4 470	5 310	3 730	4 420	7 310	8 430	6 090	7 020	10 400	11 800	8 700	9 830
24	545	690	455	575	2 140	2 650	1 780	2 210	4 560	5 450	3 800	4 540	7 520	8 750	6 270	7 290	10 800	12 300	9 030	10 300
26	545	690	455	575	2 150	2 670	1 790	2 230	4 630	5 580	3 850	4 640	7 700	9 030	6 420	7 530	11 200	12 800	9 320	10 700
28	545	690	455	575	2 160	2 700	1 800	2 250	4 680	5 680	3 900	4 740	7 860	9 280	6 550	7 740	11 500	13 300	9 570	11 100
30	545	690	455	575	2 170	2 710	1 810	2 260	4 730	5 770	3 940	4 810	7 990	9 500	6 660	7 920	11 800	13 700	9 800	11 400
Very great...	545	690	455	575	2 180	2 770	1 820	2 310	4 910	6 230	4 090	5 190	8 730	11 100	7 270	9 230	13 600	17 300	11 400	14 400

NOTE.— Ordinary Pipe Laying is pipe laying in accordance with customary good practice in tile-drain construction, whereby the underside of the pipe is well bedded on soil for 60 to 90 deg. of the circumference.

First-Class Pipe Laying is pipe laying in accordance with the best customary practice in pipe-sewer construction, whereby the entire underside of the pipe is very thoroughly bedded on soil and the entire pipe is surrounded by well-compacted soil, under the direction of an inspector constantly present on the work.

When pipe is laid in a Concrete or Other Permanent Masonry Cradle, strong enough to carry the entire load to the sub-base without breaking and large enough to prevent material settlement, the standard strengths for all dimensions of ditches and all filling materials shall be those specified for Standard Drain Tile in Table I.

Strength Test Requirements when Manufacturer is held Responsible for Cracking in Ditches.

36. The manufacturer or other seller shall not be held responsible for cracking of drain tile in ditches unless by special agreement in advance, and in any event his obligation shall be held to be discharged by the delivery of drain tile having the minimum ordinary supporting strengths specified in Table II; and, if it is not otherwise specified in advance by the purchaser, tile shall be supplied of the strengths specified for clay ditch filling, for "ordinary" pipe laying and for widths of ditch at the level of the top of the tile equal to 0.5 ft. greater than the outside diameters of the tile. The purchaser shall furnish to the manufacturer or other seller complete information, in advance of receiving bids, as to the number of linear feet of drain tile of each diameter required for each different depth of ditch, measured to the nearest foot from the surface of the ground to the top of the tile.

V. VISUAL INSPECTION.

Visual Inspection and its Purposes.

37. All drain tile shall be given a thorough visual inspection at the trench by a competent inspector employed by the purchaser. The purposes of the visual inspection shall be: (1) to cull and reject imperfect individual tiles; and (2) to determine whether the tiles, independently of meeting the chemical and the physical test requirements, comply with the specifications of general properties, especially as stated hereinafter.

Shape.

38. All drain tile shall be of approximately circular cross-section, except when otherwise specified in advance. They shall be approximately straight, except in the case of special connections. The ends shall be so regular and smooth as readily to admit of making close joints by turning and pressing together adjoining tile.

Nominal Sizes.

39. The sizes of drain tile shall be designated by their interior diameters.

Minimum Lengths.

40. Drain tile smaller than 12 in. in diameter shall have a minimum length of 12 in. Tile of from 12 to 30 in. in diameter, inclusive, shall have lengths not less than the diameters. Tile larger than 30 in. in diameter shall have a minimum length of 30 in.

Structure.

41. Drain tile shall be substantially uniform in structure throughout, and the inspector shall investigate this property by examining fractured surfaces.

42. Drain tile shall give a clear ring when stood on end and while dry tapped with a light hammer. **Ring.**

43. The inspector may use the color of drain tile as a general guide in sorting and inspecting, but he shall first so familiarize himself with the raw materials and the processes used in the manufacture of the particular tile in question as to be competent to interpret the true meaning of variations in their color. **Color.**

44. Drain tile shall be reasonably smooth on the inside. **Inside Smoothness.**

45. Drain tile shall be free from cracks and checks extending into the body of the tile in such a manner as to decrease the strength appreciably. Tile shall not be chipped or broken in such a manner as to decrease their strength materially or to admit earth into the drain. **Cracks, Checks, Chips and Broken Pieces.**

TABLE III.—DISTINCTIVE GENERAL PHYSICAL PROPERTIES OF DIFFERENT CLASSES OF DRAIN TILE.

Physical Properties Specified.	Farm Drain Tile.	Standard Drain Tile.	Extra-Quality Drain Tile.
Allowable variation of average diameter below specified diameter, per cent	5	3	3
Allowable variation between maximum and minimum diameters of same tile, or average diameters of adjoining tile, percentage of thickness of wall	85	75	65
Allowable variation from straightness, percentage of length	5	3	3
Allowable thickness of exterior blisters, lumps and flakes which do not weaken tile and are few in number, percentage of thickness of wall	25	20	15
Allowable diameters of above blisters, lumps and flakes, percentage of internal diameter	20	15	10
General Inspection	Careful.	Rigid.	Very rigid.

46. All drain tile shall be sufficiently "vitrified" or "hard-burned" to afford the degree of supporting strength, percentage of absorption, and resistance to freezing and thawing specified in the physical test requirements prescribed in Table I. **Use of the Terms Vitrified and Hard-Burned.**

47. The manufacturer or other seller may appeal from decisions of the inspector on questions of strength or structure when such decisions are based on visual inspection alone, in which case the point at issue shall be determined by standard physical tests, the cost of which shall be paid by the appellant, if the inspector was right, or by the purchaser if his inspector was in error. **Appeal from results of Visual Inspection.**

Additional Distinctive Physical Characteristics. 48. Drain tile of the different classes shall, in addition to all requirements heretofore specified, have the distinctive physical characteristic prescribed in Table III.

VI. TESTING, INSPECTION AND REJECTION

Making and Reporting Tests. 49. All tests shall be made by experts employed by the purchaser. Full reports of all tests shall be furnished the manufacturer or other seller on his request. Tests shall be made and reported promptly.

Expense of Making Tests. 50. The purchaser shall pay the expense of making all tests except as otherwise specified in Sections 9, 34, 35, 47 and 52.

Number of Tests 51. The number of standard tests to be made shall be determined by the purchaser.

General Tests and Inspection at Factory. 52. In all contracts for ten or more carloads of tile, preliminary general tests and inspection shall be made at the factory by the purchaser upon demand of the manufacturer or other seller. The expense of such tests and inspection shall be paid by the manufacturer or other seller.

Inspector. 53. The inspector shall be employed by the purchaser.

Inspection. 54. The manufacturer or other seller of the drain tile shall afford the inspector all reasonable facilities for his work, both as to the selection of specimens for tests, and as to visual inspection. Inspection shall be completed promptly.

Rejection. 55. The inspector shall plainly mark all drain tile which he rejects, and such rejected tile shall be removed promptly by the manufacturer or other seller. Upon request of the purchaser, the manufacturer or other seller shall give full account of the removal of rejected tile.

Final Report

OF THE

Joint Committee on Concrete and Reinforced Concrete

Preliminary Draft Prepared and Submitted by the Secretary, October 27, 1908
Amended and Adopted by Letter Ballot of the Committee, December 20 1908
Revised and Brought Up to Date, November 20, 1912.
Final Report Adopted by the Committee, July 1, 1916.

AFFILIATED COMMITTEES

OF THE

American Society of Civil Engineers,
American Society for Testing Materials,
American Railway Engineering Association,
Portland Cement Association,
American Concrete Institute.

JULY 1, 1916.

CHAPTER I.

Introduction.

The Joint Committee on Concrete and Reinforced Concrete was formed by the union of Special Committees appointed in 1903 and 1904 by the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering and Maintenance of Way Association (now the American Railway Engineering Association), and the Association of American Portland Cement Manufacturers (now the Portland Cement Association). In 1915 there was added a Special Committee appointed by the American Concrete Institute at the invitation of the Joint Committee.

The present organization and membership of the Joint Committee is as follows:

Officers.

Chairman—Joseph R. Worcester.
Vice-Chairman—Emil Swensson.
Secretary—Richard L. Humphrey.

Members.

american society of civil engineers.

John E. Greiner, Consulting Engineer, Baltimore, Md.

William K. Hatt, Professor of Civil Engineering, Purdue University, Lafayette, Ind.

Olaf Hoff, Consulting Engineer, 149 Broadway, New York, N. Y.

Richard L. Humphrey, Consulting Engineer, 805 Harrison Building, Philadelphia, Pa.

Robert W. Lesley, Past-President, Association of American Portland Cement Manufacturers, Pennsylvania Building, Philadelphia, Pa.

Emil Swensson, Consulting Engineer, 925 Frick Building, Pittsburgh, Pa.

Arthur N. Talbot, Professor of Municipal and Sanitary Engineering, University of Illinois, Urbana, Ill.

Joseph R. Worcester, Consulting Engineer, 79 Milk Street, Boston, Mass.

AMERICAN SOCIETY FOR TESTING MATERIALS.

William B. Fuller, Consulting Engineer, 150 Nassau Street, New York, N. Y.

Edward E. Hughes, General Manager, Franklin Steel Works, Franklin, Pa.

Richard L. Humphrey, Consulting Engineer, 805 Harrison Building, Philadelphia, Pa.

Albert L. Johnson, Vice-President and General Manager, Corrugated Bar Company, Mutual Life Building, Buffalo, N. Y.

Robert W. Lesley, Past-President, Association of American Portland Cement Manufacturers, Pennsylvania Building, Philadelphia, Pa.

Gaetano Lanza, The Montevista, Sixty-third and Oxford Streets, Overbrook, Philadelphia, Pa.

Leon S. Moisseiff, Consulting Engineer, 69 Wall Street, New York, N. Y.

Henry H. Quimby, Chief Engineer, Department of City Transit, Bourse Building, Philadelphia, Pa.

Sanford E. Thompson, Consulting Engineer, 136 Federal Street, Boston, Mass.

Frederick R. Turneaure, Dean of College of Mechanics and Engineering, University of Wisconsin, Madison, Wis.

Samuel Tobias Wagner, Chief Engineer, Philadelphia and Reading Railway Company, Reading Terminal, Philadelphia, Pa.

George S. Webster, Director, Wharves, Docks and Ferries, Bourse Building, Philadelphia, Pa.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.

H. A. Cassil, Division Engineer, Baltimore and Ohio Railway Company, New Castle, Pa.

Frederick E. Schall, Bridge Engineer, Lehigh Valley Railway Company, South Bethlehem, Pa.

Frederick P. Sisson, Assistant Engineer, Grand Trunk Railway, Detroit, Mich.

Joseph J. Yates, Bridge Engineer, Central Railroad of New Jersey, 143 Liberty Street, New York, N. Y.

PORTLAND CEMENT ASSOCIATION.

Norman D. Fraser, President, Chicago Portland Cement Company, 30 North La Salle Street, Chicago, Ill.

Robert E. Griffith, Vice-President, Giant Portland Cement Company, Pennsylvania Building, Philadelphia, Pa.

Spencer B. Newberry, President, Sandusky Portland Cement Company, Engineers' Building, Cleveland, Ohio.

AMERICAN CONCRETE INSTITUTE.

Edward Godfrey. Structural Engineer, Robert W. Hunt & Co., Monongahela Building, Pittsburgh, Pa.

Egbert J. Moore, Chief Engineer, Turner Construction Company, 11 Broadway, New York, N. Y.

Leonard C. Wason, President, Aberthaw Construction Company, 27 School Street, Boston, Mass.

Progress reports by the Joint Committee were presented to the parent societies in 1909 and 1912. The report presented in 1912 has beeen printed by the American Society of Civil Engineers, the American Society for Testing Materials and the American Railway Engineering Association, and reference to that report may be made for details regarding the earlier work of the Joint Committee, a historical sketch of the introduction of concrete and reinforced concrete, and a bibliography of authorities upon which the report was based.

The Committee has been much gratified at the reception accorded its 1912 report, and realizes the responsibility which rests upon it because of the very extensive adoption of its recommendations in current practice in this country. The members of the Committee are well aware of the incompleteness of that report, and even now they are unable to pass judgment upon some matters not dealt with in the present report.

Since 1912 the Committee has continued its study of the subject, has followed the working out of its recommendations in actual construction, has weighed arguments and criticisms which have come to its attention, and has considered new experimental

data. While the Committee sees no reason for making any fundamental changes, the recommendations of its previous report have been revised to some extent, and considerable new material has been added upon subjects not previously touched. There are some subjects upon which experimentation is still in progress, and the art of concrete and reinforced concrete will be advancing for many years to come.

While this report deals with every kind of stress to which concrete is subjected and includes all ordinary conditions of proportioning and handling, it does not go into all types of construction nor all the applications to which concrete and reinforced concrete may be put. The report is not a specification but may be used as a basis for specifications. In their use concrete and reinforced concrete involve the exercise of good judgment to a greater degree than do any other building materials. Rules cannot produce or supersede judgment; on the contrary, judgment should control the interpretation and application of rules.

The Committee has not attempted in every case to present rigidly scientific methods of analysis in dealing with stresses, but has aimed to furnish rules which will lead to safe results sufficiently close for ordinary design.

The Committee presumes that the application of the recommendations in this report to the design of any structure will be made only by persons having an adequate knowledge of the principles of structural design. Only persons with such knowledge and experience should be called upon to design reinforced concrete structures.

The Joint Committee has reached the conclusion that, with this effort to express the present state of the art, it would be desirable for it to withdraw from the field. This action has been taken in the hope that a work similar to that which the Committee has attempted to perform will again be undertaken, within a reasonable term of years, in order that there may be some authoritative body to consider and pass upon newly acquired knowledge and information gleaned from experience. The Committee feels certain, however, that it would be for the better interest of the profession to entrust this work to other hands rather than to continue the present organization with this object in view.

CHAPTER II.

ADAPTABILITY OF CONCRETE AND REINFORCED CONCRETE.

The adaptability of concrete and reinforced concrete for engineering structures or parts thereof, is so well established that they are recognized materials of construction. When properly used, they have proved satisfactory for those purposes for which their qualities make them particularly suitable.

1. USES.

Plain concrete is well adapted for structures in which the principal stresses are compressive, such as:—foundations, dams, retaining and other walls, tunnels, piers, abutments, and, in many cases, arches.

By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in various structures and structural forms. This combination of concrete and metal is particularly advantageous in structural members subject to both compression and tension, and in columns where, although the main stresses are compressive, there is also cross-bending.

Metal reinforcement may also be used to advantage to distribute and minimize cracks due to shrinkage and temperature changes.

2. PRECAUTIONS.

Failures of reinforced concrete structures have been due usually to some one or more of the following causes:

Defective design, poor material, faulty execution, or premature removal of forms.

To prevent failures or otherwise unsatisfactory results, the following precautions should be taken:

The computations and assumptions on which the design is based should be in accordance with the established principles of mechanics. The unit stresses and details of the design should conform to accepted good practice. Materials used for the concrete as well as for the reinforcement should be carefully inspected and tested, special attention being given to the testing of the sand, as poor sand has proved a frequent cause

of failure. The measuring and combining of the materials which go to make up the concrete, and the placing of the concrete in the forms, should be under the supervision of experienced men. The metal for reinforcement should be of a quality conforming to standard specifications. Care should be taken to obtain good bond between different fills of concrete, to prevent concrete from freezing before the cement has set, to have the materials thoroughly mixed, to avoid too wet or too dry a consistency, and to have the forms cleaned before concrete is placed.

The computations should include all details; even minor details may be of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connection between the component parts so that they cannot be displaced. As the connections between reinforced concrete members are frequently a source of weakness, the design should include a detailed study of such connections.

The concrete should be rigidly supported until it has developed sufficient strength to carry imposed loads. The most careful and experienced inspection is necessary to determine when the concrete has set sufficiently for it to be safe to remove forms. Frozen concrete frequently has been mistaken for properly set concrete.

3. DESIGN AND SUPERVISION.

The execution of the work should not be separated from the design, as intelligent supervision and successful execution can be expected only when both functions are combined. It is desirable, therefore, that the engineer who prepares the design and specifications should have supervision of the execution of the work.

The Committee recommends the following practice for the purpose of fixing the responsibility and providing for adequate supervision during construction:

(*a*) Before work is commenced, complete plans and specifications should be prepared, giving the dead and live loads, wind and impact, if any, and working stresses, showing the general arrangement and all details. The plans should show the size, length, location of points of bending, and exact position of all reinforcement, including stirrups, ties, hooping and splicing.

(*b*) The specifications should state the qualities of the materials and the proportions in which they are to be used.

(*c*) The strength which the concrete is expected to attain after a definite period should be stated in the specifications.

(*d*) Inspection during construction should be made by competent inspectors selected by and under the supervision of the engineer, and should cover the following:

1. Materials.
2. Construction and erection of the forms and supports.
3. Sizes, shapes, arrangement, position and fastening of the reinforcement.
4. Proportioning, mixing, consistency, and placing of the concrete.
5. Strength of the concrete by tests of standard test pieces made on the work.
6. Whether the concrete is sufficiently hardened before the forms and supports are removed.
7. Protection from injury of all parts of the structure.
8. Comparison of dimensions of all parts of the finished structure with the plans.

(*e*) Load tests on portions of the finished structure should be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired, or where there is any doubt as to the sufficiency of the design. The loading should be carried to such a point that the calculated stresses under such loading shall be one and three-quarters times the allowed working stresses, and such loads should cause no injurious permanent deformations. Load tests should not be made before the concrete has been in place sixty days.

4. DESTRUCTIVE AGENCIES.

(*a*) *Corrosion of Metal Reinforcement.*—Tests and experience indicate that steel sufficiently embedded in good concrete is well protected against corrosion, no matter whether located above or below water level. It is recommended that such protection be not less than 1 in. in thickness. If the concrete is porous so as to be readily permeable by water, as when the concrete is

laid with a very dry consistency, the metal may corrode on account of the presence of moisture and air.

(b) *Electrolysis.*—The experimental data available on this subject seem to show that while reinforced concrete structures may, under certain conditions, be injured by the flow of electric current in either direction between the reinforcing material and the concrete, such injury is generally to be expected only where voltages are considerably higher than those which usually occur in concrete structures in practice. If the iron be positive, trouble may manifest itself by corrosion of the iron accompanied by cracking of the concrete, and, if the iron be negative, there may be a softening of the concrete near the surface of the iron, resulting in a destruction of the bond. The former, or anode effect, decreases much more rapidly than the voltage, and almost if not quite disappears at voltages that are most likely to be encountered in practice. The cathode effect, on the other hand, takes place even under very low voltages, and is therefore more important from a practical standpoint than that of the anode.

Structures containing salt or calcium chloride, even in very small quantities, are very much more susceptible to the effects of electric currents than normal concrete, the anode effect progressing much more rapidly in the presence of chlorine, and the cathode effect being greatly increased by the presence of an alkali metal.

There is great weight of evidence to show that normal reinforced concrete structures free from salt are in very little danger under most practical conditions, while non-reinforced concrete structures are practically immune from electrolysis troubles.

(c) *Sea Water.*—The data available concerning the effect of sea water on concrete or reinforced concrete are limited and inconclusive. Sea walls out of the range of frost action have been standing for many years without apparent injury. In many places serious disintegration has taken place. This has occurred chiefly between low and high tide levels and is due, evidently, in part to frost. Chemical action also appears to be indicated by the softening of the mortar. To effect the best resistance to sea water, the concrete must be proportioned, mixed and placed so as to prevent the penetration of sea water into the mass or through the joints. The aggregates should be carefully selected,

graded and proportioned with the cement so as to secure the maximum possible density; the concrete should be thoroughly mixed; the joints between old and new work should be made watertight; and the concrete should be kept from exposure to sea water until it is thoroughly hard and impervious.

(*d*) *Acids.*—Dense concrete thoroughly hardened is affected appreciably only by acids which seriously injure other materials. Substances like manure that contain acids may injuriously affect green concrete, but do not affect concrete that is thoroughly hardened.

(*e*) *Oils.*—Concrete is unaffected by such mineral oils as petroleum and ordinary engine oils. Oils which contain fatty acids produce injurious effects, forming compounds with the lime which may result in a disintegration of the concrete in contact with them.

(*f*) *Alkalies.*—The action of alkalies on concrete is problematical. In the reclamation of arid land where the soil is heavily charged with alkaline salts it has been found that concrete, stone, brick, iron and other materials are injured under certain conditions. It would seem that at the level of the ground water in an extremely dry atmosphere such structures are disintegrated, through the rapid crystallization of the alkaline salts, resulting from the alternate wetting and drying of the surface. Such destructive action can be prevented by the use of a protective coating and is minimized by securing a dense concrete.

CHAPTER III.

MATERIALS.

The quality of all the materials is of paramount importance The cement and also the aggregates should be subject to definite requirements and tests.

1. CEMENT.

There are available for construction purposes Portland, Natural and Puzzolan or Slag cements.

(*a*) *Portland Cement* is the product obtained by finely pulverizing clinker produced by calcining to incipient fusion, an

intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

It has a definite chemical composition varying within comparatively narrow limits.

Portland cement only should be used in reinforced concrete construction or in any construction that will be subject to shocks, vibrations, or stresses other than direct compression.

(b) *Natural Cement* is the finely pulverized product resulting from the calcination of an argillaceous limestone at a temperature only sufficient to drive off the carbonic acid gas.

Although the limestone must have a certain composition, this composition may vary within much wider limits than in the case of Portland cement. Natural cement does not develop its strength as quickly nor is it as uniform in composition as Portland cement.

Natural cement may be used in massive masonry where weight rather than strength is the essential feature.

Where economy is the governing factor a comparison may be made between the use of natural cement and a leaner mixture of Portland cement that will develop the same strength.

(c) *Puzzolan or Slag Cement* is the product resulting from finely pulverizing a mechanical mixture of granulated basic blast-furnace slag and hydrated lime.

Puzzolan cement is not nearly as strong, uniform, or reliable as Portland or natural cement, is not used extensively, and never in important work; it should be used only for unimportant foundation work underground where it is not exposed to air or running water.

(d) *Specifications.*—The cement should meet the requirements of the specifications and methods of tests for Portland cement, which are the result of the joint labors of special committees of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering Association, and other affiliated organizations, and the United States Government.

2. AGGREGATES.

Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for

the purpose of determining the quality and grading necessary to secure maximum density[1] or a minimum percentage of voids. Bank gravel should be separated by screening into fine and coarse aggregates and then used in the proportions to be determined by density tests.

(*a*) *Fine Aggregate* should consist of sand, or the screenings of gravel or crushed stone, graded from fine to coarse, and passing when dry a screen having $\frac{1}{4}$-in. diameter holes;[2] it preferably should be of siliceous material, and not more than 30 per cent by weight, should pass a sieve having 50 meshes per linear inch; it should be clean, and free from soft particles, lumps of clay, vegetable loam or other organic matter.

Fine aggregate should always be tested for strength. It should be of such quality that mortar composed of one part Portland cement and three parts fine aggregate by weight when made into briquettes, prisms or cylinders will show a tensile or compressive strength, at an age of not less than 7 days, at least equal to the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.[3] If the aggregate be of poorer quality, the proportion of cement should be increased to secure the desired strength. If the strength developed by the aggregate in the 1:3 mortar is less than 70 per cent of the strength of the Ottawa-sand mortar, the material should be rejected. In testing aggregates care should be exercised to avoid the removal of any coating on the grains, which may affect the strength; bank sands should not be dried before being made into mortar, but should contain natural moisture. The percentage of moisture may be determined upon a separate sample for correcting weight. From 10 to 40 per cent more water may be required in mixing bank or artificial sands than for standard Ottawa sand to produce the same consistency.

(*b*) *Coarse Aggregate* should consist of gravel or crushed stone which is retained on a screen having $\frac{1}{4}$-in. diameter holes, and

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[1] A convenient coefficient of density is the ratio of the sum of the volumes of solid particles contained in a unit volume to the total unit volume.

[2] If the dividing size between the fine and coarse aggregate is less or greater than one-quarter inch, allowance should be made in grading and proportioning.

[3] A natural sand obtained at Ottawa, Illinois, passing a screen having 20 meshes and retained on a screen having 30 meshes per linear inch; prepared and furnished by the Ottawa Silica Company, for 2 cents per pound f. o. b. cars, Ottawa, Illinois.

should be graded from the smallest to the largest particles; it should be clean, hard, durable, and free from all deleterious matter. Aggregates containing dust and soft, flat or elongated particles, should be excluded. The Committee does not feel warranted in recommending the use of blast furnace slag as an aggregate, in the absence of adequate data as to its value, especially in reinforced concrete construction. No satisfactory specifications or methods of inspection have been developed that will control its uniformity and ensure the durability of the concrete in which it is used.

The aggregate must be small enough to produce with the mortar a homogeneous concrete of sluggish consistency which will pass readily between and easily surround the reinforcement and fill all parts of the forms. The maximum size of particles is variously determined for different types of construction from that which will pass a ½-in. ring to that which will pass a 1½-in. ring.

For concrete in large masses the size of the coarse aggregate may be increased, as a large aggregate produces a stronger concrete than a fine one; however it should be noted that the danger of separation from the mortar becomes greater as the size of the coarse aggregate increases.

Cinder concrete should not be used for reinforced concrete structures except in floor slabs not exceeding 8-ft. span. It also may be used for fire protection purposes where not required to carry loads. The cinders used should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal or ashes.

3. WATER.

The water used in mixing concrete should be free from oil, acid, alkali, or organic matter.

4. METAL REINFORCEMENT.

The Committee recommends as a suitable material for reinforcement, steel of structural grade filling the requirements of the Specifications for Billet-Steel Concrete Reinforcement Bars of the American Society for Testing Materials.

For reinforcing slabs, small beams or minor details, or for reinforcing for shrinkage and temperature stresses, steel wire,

expanded metal, or other reticulated steel may be used, with the unit stresses hereinafter recommended.

The reinforcement should be free from flaking rust, scale, or coatings of any character which would tend to reduce or destroy the bond.

CHAPTER IV.

Mixing and Placing.

1. PROPORTIONS.

The materials should be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density, which is obtained by grading the aggregates so that the smaller particles fill the spaces between the larger thus reducing the voids in the aggregate to the minimum.

(*a*) *Unit of Measure.*—The measurement of the fine and coarse aggregates should be by loose volume. The unit of measure should be a bag of cement, containing 94 lb. net, which should be considered the equivalent of one cubic foot.

(*b*) *Relation of Fine and Coarse Aggregates.*—The fine and coarse aggregates should be used in such proportions as will secure maximum density. These proportions should be carefully determined by density experiments and the grading of the fine and coarse aggregates should be uniformly maintained, or the proportions changed to meet the varying sizes.

(*c*) *Relation of Cement and Aggregates.*—For reinforced concrete construction, one part of cement to a total of six parts of fine and coarse aggregates measured separately should generally be used. For columns, richer mixtures are preferable. In massive masonry or rubble concrete a mixture of 1 : 9 or even 1 : 12 may be used.

These proportions should be determined by the strength or other qualities required in the construction at the critical period of use. Experience and judgment based on observation and tests of similar conditions in similar localities are excellent guides as to the proper proportions for any particular case.

In important construction, advance tests should be made on concrete composed of the materials to be used in the work.

These tests should be made by standardized methods to obtain uniformity in mixing, proportioning and storage, and in case the results do not conform to the requirements of the work, aggregates of a better quality or more cement should be used to obtain the desired quality of concrete.

2. MIXING.

The mixing of concrete should be thorough, and continue until the mass is uniform in color and homogeneous. As the maximum density and greatest strength of a given mixture depend largely on thorough and complete mixing, it is essential that this part of the work should receive special attention and care.

Inasmuch as it is difficult to determine, by visual inspection, whether the concrete is uniformly mixed, especially where aggregates having the color of cement are used, it is essential that the mixing should occupy a definite period of time. The minimum time will depend on whether the mixing is done by machine or hand.

(*a*) *Measuring Ingredients.*—Methods of measurement of the various ingredients should be used which will secure at all times separate and uniform measurements of cement, fine aggregate, coarse aggregate, and water.

(*b*) *Machine Mixing.*—The mixing should be done in a batch machine mixer of a type which will ensure the uniform distribution of the materials throughout the mass, and should continue for the minimum time of one and one-half minutes after all the ingredients are assembled in the mixer. For mixers of two or more cubic yards capacity, the minimum time of mixing should be two minutes. Since the strength of the concrete is dependent upon thorough mixing, a longer time than this minimum is preferable. It is desirable to have the mixer equipped with an attachment for automatically locking the discharging device so as to prevent the emptying of the mixer until all the materials have been mixed together for the minimum time required after they are assembled in the mixer. Means should be provided to prevent aggregates being added after the mixing has commenced. The mixer should also be equipped with water storage, and an automatic measuring device which can be locked is desirable. It is also desirable to equip the mixer with a device recording the

revolutions of the drum. The number of revolutions should be so regulated as to give at the periphery of the drum a uniform speed; about 200 ft. per minute seems to be the best speed in the present state of the art.

(*c*) *Hand Mixing*.—Hand mixing should be done on a water-tight platform and especial precautions taken after the water has been added, to turn all the ingredients together at least six times, and until the mass is homogeneous in appearance and color.

(*d*) *Consistency*.—The materials should be mixed wet enough to produce a concrete of such a consistency as will flow sluggishly into the forms and about the metal reinforcement when used, and which, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar. The quantity of water is of the greatest importance in securing concrete of maximum strength and density; too much water is as objectionable as too little.

(*e*) *Retempering*.—The remixing of mortar or concrete that has partly set should not be permitted.

3. PLACING CONCRETE.

(*a*) *Methods*.—Concrete after the completion of the mixing should be conveyed rapidly to the place of final deposit; under no circumstances should concrete be used that has partly set.

Concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients are in their proper place. Special care should be exercised to prevent the formation of laitance; where laitance has formed it should be removed, since it lacks strength, and prevents a proper bond in the concrete.

Before depositing concrete, the reinforcement should be carefully placed in accordance with the plans. It is essential that adequate means be provided to hold it in its proper position until the concrete has been deposited and compacted; care should be taken that the forms are substantial and thoroughly wetted (except in freezing weather) or oiled and that the space to be occupied by the concrete is free from débris. When the placing of concrete is suspended, all necessary grooves for joining future work should be made before the concrete has set.

When work is resumed, concrete previously placed should be roughened, cleansed of foreign material and laitance, thoroughly wetted and then slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

The surfaces of concrete exposed to premature drying should be kept covered and wet for a period of at least seven days.

Where concrete is conveyed by spouting, the plant should be of such a size and design as to ensure a practically continuous stream in the spout. The angle of the spout with the horizontal should be such as to allow the concrete to flow without a separation of the ingredients; in general an angle of about 27 deg. or one vertical to two horizontal is good practice. The spout should be thoroughly flushed with water before and after each run. The delivery from the spout should be as close as possible to the point of deposit. Where the discharge must be intermittent, a hopper should be provided at the bottom. Spouting through a vertical pipe is satisfactory when the flow is continuous; when it is unchecked and discontinuous it is highly objectionable unless the flow is checked by baffle plates.

(*b*) *Freezing Weather.*—Concrete should not be mixed or deposited at a freezing temperature, unless special precautions are taken to prevent the use of materials covered with ice crystals or containing frost, and to prevent the concrete from freezing before it has set and sufficiently hardened.

As the coarse aggregate forms the greater portion of the concrete, it is particularly important that this material be warmed to well above the freezing point.

The enclosing of a structure and the warming of the space inside the enclosure is recommended, but the use of salt to lower the freezing point is not recommended.

(*c*) *Rubble Concrete.*—Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced by the use of clean stones, saturated with water, thoroughly embedded in and entirely surrounded by concrete.

(*d*) *Under Water.*—In placing concrete under water it is essential to maintain still water at the place of deposit. With careful inspection the use of tremies, properly designed and operated, is a satisfactory method of placing concrete through water. The concrete should be mixed very wet (more so than is ordinarily

permissible) so that it will flow readily through the tremie and into place with practically a level surface.

The coarse aggregate should be smaller than ordinarily used, and never more than 1 in. in diameter. The use of gravel facilitates mixing and assists the flow. The mouth of the tremie should be buried in the concrete so that it is at all times entirely sealed and the surrounding water prevented from forcing itself into the tremie; the concrete will then discharge without coming in contact with the water. The tremie should be suspended so that it can be lowered quickly when it is necessary either to choke off or prevent too rapid flow; the lateral flow preferably should be not over 15 ft.

The flow should be continuous in order to produce a monolithic mass and to prevent the formation of laitance in the interior.

In case the flow is interrupted it is important that all laitance be removed before proceeding with the work.

In large structures it may be necessary to divide the mass of concrete into several small compartments or units, to permit the continuous filling of each one. With proper care it is possible in this manner to obtain as good results under water as in the air.

A less desirable method is the use of the drop bottom bucket. Where this method is used, the bottom of the bucket should be released when in contact with the surface of the place of deposit.

CHAPTER V.

FORMS.

Forms should be substantial and unyielding, in order that the concrete may conform to the design, and be sufficiently tight to prevent the leakage of mortar.

It is vitally important to allow sufficient time for the proper hardening of the concrete, which should be determined by careful inspection before the forms are removed.

Many conditions affect the hardening of concrete, and the proper time for the removal of the forms should be determined by some competent and responsible person.

It may be stated in a general way that forms should remain in place longer for reinforced concrete than is required for plain

or massive concrete, and longer for horizontal than is required for vertical members.

In general it may be considered that concrete has hardened sufficiently when it has a distinctive ring under the blow of a hammer, but this test is not reliable, if there is a possibility that the concrete is frozen.

CHAPTER VI.

DETAILS OF CONSTRUCTION.

1. JOINTS.

(*a*) *In Concrete.*—It is desirable to cast an entire structure at one operation, but as this is not always possible, especially in large structures, it is necessary to stop the work at some convenient point. This should be selected so that the resulting joint may have the least possible effect on the strength of the structure. It is therefore recommended that the joint in columns be made flush with the lower side of the girders, or in flat slab construction at the bottom of the flare of the column head; that the joints in girders be at a point midway between supports, unless a beam intersect a girder at this point, in which case the joint should be offset a distance equal to twice the width of the beam; and that the joints in the members of a floor system should in general be made at or near the center of the span.

Joints in columns should be perpendicular to the axis, and in girders, beams, and floor slabs, perpendicular to the plane of their surfaces. When it is necessary to provide for shear at right angles to the axis, it is permissible to incline the plane of the joint as much as 30 deg. from the perpendicular. Joints in arch rings should be on planes as nearly radial as practicable.

Before placing the concrete on top of a freshly poured column a period of at least two hours should be allowed for the settlement and shrinkage.

Shrinkage and contraction joints may be necessary to concentrate cracks due to temperature in smooth even lines. The number of these joints which should be determined and provided

for in the design will depend on the range of temperature to which the concrete will be subjected, and on the amount and position of the reinforcement. In massive work, such as retaining walls, abutments, etc., built without reinforcement, contraction joints should be provided, at intervals of from 25 to 50 ft. and with reinforcement from 50 to 80 ft.; the smaller the height and thickness, the closer the spacing. The joints should be tongued and grooved to maintain the alignment in case of unequal settlement. A groove may be formed in the surface as a finish to vertical joints.

Shrinkage and contraction joints should be lubricated by an application of petroleum oil or a similar material to permit a free movement when the concrete expands or contracts.

The movement of the joint due to expansion and contraction may be facilitated by the insertion of a sheet of copper, zinc, or even tarred paper.

(*b*) *In Reinforcement.*—Wherever it is necessary to splice tension reinforcement the length of lap should be determined on the basis of the safe bond stress, the stress in the bar and the shearing resistance of the concrete at the point of splice; or a connection should be made between the bars of sufficient strength to carry the stress. Splices at points of maximum stress in tension should be avoided. In columns, bars more than $\frac{3}{4}$ in. in diameter not subject to tension should have their ends properly squared and butted together in suitable sleeves; smaller bars may be lapped as indicated for tension reinforcement. At foundations bearing plates should be provided for supporting the bars, or the bars may be carried into the footing a sufficient distance to transmit the stress in the steel to the concrete by means of the bearing and the bond resistance. In no case should reliance be placed upon the end bearing of bars on concrete.

2. SHRINKAGE AND TEMPERATURE CHANGES.

The stresses resulting from shrinkage due to hardening and contraction from temperature changes are important in monolithic construction, and unless cared for in the design will produce objectionable cracks; cracks cannot be entirely prevented but the effects can be minimized.

Large cracks, produced by quick hardening or wide ranges of temperature, can be broken up to some extent into small cracks by placing reinforcement in the concrete; in long, continuous lengths of concrete, it is better to provide shrinkage joints at points in the structure where they will do little or no harm. Reinforcement permits longer distances between shrinkage joints than when no reinforcement is used.

Provision for shrinkage should be made where small or thin masses are joined to larger or thicker masses; at such places the use of fillets similar to those used in metal castings, but proportionally larger, is recommended.

Shrinkage cracks are likely to occur at points where fresh concrete is joined to that which is set, and hence in placing the concrete, construction joints should be made, as described in Chapter VI, Sect. 1, or if possible, at points where joints would naturally occur in dimension stone masonry.

3. FIREPROOFING.

Concrete, because incombustible and of a low rate of heat conductivity, is highly efficient and admirably adapted for fireproofing purposes. This has been demonstrated by experience and tests.

The dehydration of concrete probably begins at about 500° F. and is completed at about 900° F., but experience indicates that the volatilization of the water absorbs heat from the surrounding mass, which, together with the resistance of the air cells, tends to increase the heat resistance of the concrete, so that the process of dehydration is very much retarded. The concrete that is actually affected by fire and remains in position affords protection to that beneath it.

The thickness of the protective coating should be governed by the intensity and duration of a possible fire and the rate of heat conductivity of the concrete. The question of the rate of heat conductivity of concrete is one which requires further study and investigation before a definite rate for different classes of concrete can be fully established. However, for ordinary conditions it is recommended that the metal be protected by a minimum of 2 in. of concrete on girders and columns, $1\frac{1}{2}$ in. on beams, and 1 in. on floor slabs.

Where fireproofing is required and not otherwise provided in monolithic concrete columns, it is recommended that the concrete to a depth of $1\frac{1}{2}$ in. be considered as protective covering and not included in the effective section.

The corners of columns, girders, and beams should be beveled or rounded, as a sharp corner is more seriously affected by fire than a round one; experience shows that round columns are more fire resistive than square.

4. WATERPROOFING.

Many expedients have been resorted to for rendering concrete impervious to water. Experience shows, however, that when mortar or concrete is proportioned to obtain the greatest practicable density and is mixed to the proper consistency (Chap. IV, Sect. 2 *d*), the resulting mortar or concrete is impervious under moderate pressure.

On the other hand, concrete of dry consistency is more or less pervious to water, and, though compounds of various kinds have been mixed with the concrete or applied as a wash to the surface, in an effort to offset this defect, these expedients have generally been disappointing, for the reason that many of these compounds have at best but temporary value, and in time lose their power of imparting impermeability to the concrete.

In the case of subways, long retaining walls and reservoirs, provided the concrete itself is impervious, cracks may be so reduced by horizontal and vertical reinforcement properly proportioned and located, that they will be too minute to permit leakage, or will be closed by infiltration of silt.

Asphaltic or coal-tar preparations applied either as a mastic or as a coating on felt or cloth fabric, are used for waterproofing, and should be proof against injury by liquids or gases.

For retaining and similar walls in direct contact with the earth, the application of one or two coatings of hot coal-tar pitch, following a painting with a thin wash of coal tar dissolved in benzol, to the thoroughly dried surface of concrete is an efficient method of preventing the penetration of moisture from the earth.

5. SURFACE FINISH.

Concrete is a material of an individual type and should be used without effort at imitation of other building materials. One of the important problems connected with its use is the character of the finish of exposed surfaces. The desired finish should be determined before the concrete is placed, and the work conducted so as to facilitate securing it. The natural surface of the concrete in most structures is unobjectionable, but in others the marks of the forms and the flat dead surface are displeasing, making some special treatment desirable. A treatment of the surface which removes the film of cement and brings the aggregates of the concrete into relief, either by scrubbing with brushes and water before it is hard or by tooling it after it is hard, is frequently used to erase the form markings and break the monotonous appearance of the surface. Besides being more pleasing in immediate appearance such a surface is less subject to discoloration and hair cracking than is a surface composed of the cement that segregates against the forms, or one that is made by applying a cement wash. The aggregates can also be exposed by washing with hydrochloric acid diluted with from 6 to 10 parts of water. The plastering of surfaces should be avoided, for even if carefully done, it is liable to peel off under the action of frost or temperature changes.

Various effects in texture and in color can be obtained when the surface is to be scrubbed or tooled, by using aggregates of the desired size and color. For a fine grained texture a granolithic surface mixture can be made and placed against the face forms to a thickness of about 1 in. as the placing of the body of the concrete proceeds.

A smooth, even surface without form marks can be secured by the use of plastered forms, which in structures having many duplications of members can be used repeatedly; these are made in panels of expanded metal or wire mesh coated with plaster, and the joints made at edges, and closed with plaster of Paris.

CHAPTER VII.

DESIGN.

1. MASSIVE CONCRETE.

In the design of massive or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be proportioned so as to avoid tensile stresses except in slight amounts to resist indirect stresses. This will generally be accomplished in the case of rectangular shapes if the line of pressure is kept within the middle third of the section, but in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight; hence the element of weight rather than strength often determines the design. A leaner and relatively cheap concrete, therefore, will often be suitable for massive concrete structures.

It is desirable generally to provide joints at intervals to localize the effect of contraction (Chap. VI, Sect. 1).

Massive concrete is suitable for dams, retaining walls, and piers in which the ratio of length to least width is relatively small. Under ordinary conditions this ratio should not exceed four. It is also suitable for arches of moderate span.

2. REINFORCED CONCRETE.

The use of metal reinforcement is particularly advantageous in members such as beams in which both tension and compression exist, and in columns where the principal stresses are compressive and where there also may be cross-bending. Therefore the theory of design here presented relates mainly to the analysis of beams and columns.

3. GENERAL ASSUMPTIONS.

(*a*) *Loads.*—The forces to be resisted are those due to:

1. *The dead load*, which includes the weight of the structure and fixed loads and forces.

2. *The live load,* or the loads and forces which are variable. The dynamic effect of the live load will often require consideration. Allowance for the latter is preferably made by a proportionate increase in either the live load or the live load stresses. The working stresses hereinafter recommended are intended to apply to the equivalent static stresses thus determined.

   In the case of high buildings the live load on columns may be reduced in accordance with the usual practice.

(*b*) *Lengths of Beams and Columns.*—The span length for beams and slabs simply supported should be taken as the distance from center to center of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. For continuous or restrained beams built monolithically into supports the span length may be taken as the clear distance between faces of supports. Brackets should not be considered as reducing the clear span in the sense here intended, except that when brackets which make an angle of 45 deg. or more with the axis of a restrained beam are built monolithically with the beam, the span may be measured from the section where the combined depth of beam and bracket is at least one-third more than the depth of the beam. Maximum negative moments are to be considered as existing at the end of the span as here defined.

When the depth of a restrained beam is greater at its ends than at midspan and the slope of the bottom of the beam at its ends makes an angle of not more than 15 deg. with the direction of the axis of the beam at midspan, the span length may be measured from face to face of supports.

The length of columns should be taken as the maximum unstayed length.

(*c*) *Stresses.*—The following assumptions are recommended as a basis for calculations:

1. Calculations will be made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.
2. A plane section before bending remains plane after bending.

3. The modulus of elasticity of concrete in compression is constant within the usual limits of working stresses. The distribution of compressive stress in beams is therefore rectilinear.
4. In calculating the moment of resistance of beams the tensile stresses in the concrete are neglected.
5. The adhesion between the concrete and the reinforcement is perfect. Under compressive stress the two materials are therefore stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken at 15 except as modified in Chapter VIII, Section 8.
7. Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected.

It is recognized that some of the assumptions given herein are not entirely borne out by experimental data. They are given in the interest of simplicity and uniformity, and variations from exact conditions are taken into account in the selection of formulas and working stresses.

The deflection of a beam depends upon the strength and stiffness developed throughout its length. For calculating deflection a value of 8 for the ratio of the moduli will give results corresponding approximately with the actual conditions.

4. T-BEAMS.

In beam and slab construction an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used extending over the beam and well into the slab.

The slab may be considered an integral part of the beam, when adequate bond and shearing resistance between slab and web of beam is provided, but its effective width shall be determined by the following rules:

(a) It shall not exceed one-fourth of the span length of the beam;

(*b*) Its overhanging width on either side of the web shall not exceed six times the thickness of the slab.

In the design of continuous T-beams, due consideration should be given to the compressive stress at the support.

Beams in which the T-form is used only for the purpose of providing additional compression area of concrete should preferably have a width of flange not more than three times the width of the stem and a thickness of flange not less than one-third of the depth of the beam. Both in this form and in the beam and slab form the web stresses and the limitations in placing and spacing the longitudinal reinforcement will probably be controlling factors in design.

5. FLOOR SLABS SUPPORTED ALONG FOUR SIDES.

Floor slabs having the supports extending along the four sides should be designed and reinforced as continuous over the supports. If the length of the slab exceeds 1.5 times its width the entire load should be carried by transverse reinforcement.

For uniformly distributed loads on square slabs, one-half the live and dead load may be used in the calculations of moment to be resisted in each direction. For oblong slabs, the length of which is not greater than one and one-half times their width, the moment to be resisted by the transverse reinforcement may be found by using a proportion of the live and dead load equal to that given by the formula $r=\frac{l}{b}-0.5$, where l=length and b=breadth of slab. The longitudinal reinforcement should then be proportioned to carry the remainder of the load.

In placing reinforcement in such slabs account may well be taken of the fact that the bending moment is greater near the center of the slab than near the edges. For this purpose two-thirds of the previously calculated moments may be assumed as carried by the center half of the slab and one-third by the outside quarters.

Loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beams and the distribution will depend on the relative stiffness of the slab and the supporting beams. The distribution which

may be expected ordinarily is a variation of the load in the beam in accordance with the ordinates of a parabola, having its vertex at the middle of the span. For any given design, the probable distribution should be ascertained and the moments in the beam calculated accordingly.

6. CONTINUOUS BEAMS AND SLABS.

When the beam or slab is continuous over its supports, reinforcement should be fully provided at points of negative moment, and the stresses in concrete recommended in Chapter VIII, Section 4, should not be exceeded. In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:

(*a*) For floor slabs the bending moments at center and at support should be taken at $\frac{wl^2}{12}$ for both dead and live loads, where w represents the load per linear unit and l the span length.

(*b*) For beams the bending moment at center and at support for interior spans should be taken at $\frac{wl^2}{12}$, and for end spans it should be taken at $\frac{wl^2}{10}$ for center and interior support, for both dead and live loads.

(*c*) In the case of beams and slabs continuous for two spans only, with their ends restrained, the bending moment both at the central support and near the middle of the span should be taken at $\frac{wl^2}{10}$.

(*d*) At the ends of continuous beams the amount of negative moment which will be developed in the beam will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. In the ordinary cases a moment of $\frac{wl^2}{16}$ may be taken; for small beams running into heavy columns this should be increased, but not to exceed $\frac{wl^2}{12}$.

For spans of unusual length, or for spans of materially

unequal length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by (*a*) or (*b*), the negative moment at the support should not be taken as less than the values there given.

Where beams are reinforced on the compression side, the steel may be assumed to carry its proportion of stress in accordance with the ratio of moduli of elasticity, Chapter VIII, Section 8. Reinforcing bars for compression in beams should be straight and should be two diameters in the clear from the surface of the concrete. For the positive bending moment, such reinforcement should not exceed 1 per cent of the area of the concrete. In the case of cantilever and continuous beams, tensile and compressive reinforcement over supports should extend sufficiently beyond the support and beyond the point of inflection to develop the requisite bond strength.

In construction made continuous over supports it is important that ample foundations should be provided; for unequal settlements are liable to produce unsightly if not dangerous cracks. This effect is more likely to occur in low structures.

Girders, such as wall girders, which have beams framed into one side only, should be designed to resist torsional moment arising from the negative moment at the end of the beam.

7. BOND STRENGTH AND SPACING OF REINFORCEMENT.

Adequate bond strength should be provided. The formula hereinafter given for bond stresses in beams is for straight longitudinal bars. In beams in which a portion of the reinforcement is bent up near the end, the bond stress at places, in both the straight bars and the bent bars, will be considerably greater than for all the bars straight, and the stress at some point may be several times as much as that found by considering the stress to be uniformly distributed along the bar. In restrained and cantilever beams full tensile stress exists in the reinforcing bars at the point of support and the bars should be anchored in the support sufficiently to develop this stress.

In case of anchorage of bars, an additional length of bar should be provided beyond that found on the assumption of uniform bond stress, for the reason that before the bond resistance

at the end of the bar can be developed the bar may have begun to slip at another point and "running" resistance is less than the resistance before slip begins.

Where high bond resistance is required, the deformed bar is a suitable means of supplying the necessary strength. But it should be recognized that even with a deformed bar initial slip occurs at early loads, and that the ultimate loads obtained in the usual tests for bond resistance may be misleading. Adequate bond strength throughout the length of a bar is preferable to end anchorage, but, as an additional safeguard, such anchorage may properly be used in special cases. Anchorage furnished by short bends at a right angle is less effective than by hooks consisting of turns through 180 deg.

The lateral spacing of parallel bars should be not less than three diameters from center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should be not less than 1 in. The use of more than two layers is not recommended, unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or bent down. Where more than one layer is used at least all bars above the lower layer should be bent up and anchored beyond the edge of the support.

8. DIAGONAL TENSION AND SHEAR.

When a reinforced concrete beam is subjected to flexural action, diagonal tensile stresses are set up. A beam without web reinforcement will fail if these stresses exceed the tensile strength of the concrete. When web reinforcement, made up of stirrups or of diagonal bars secured to the longitudinal reinforcement, or of longitudinal reinforcing bars bent up at several points, is used, new conditions prevail, but even in this case at the beginning of loading the diagonal tension developed is taken principally by the concrete, the deformations which are developed in the concrete permitting but little stress to be taken by the web reinforcement. When the resistance of the concrete to the diagonal tension is overcome at any point in the depth of the beam, greater stress is at once set up in the web reinforcement.

For homogeneous beams the analytical treatment of diagonal tension is not very complex—the diagonal tensile stress is a function of the horizontal and vertical shearing stresses and of the horizontal tensile stress at the point considered, and as the intensity of these three stresses varies from the neutral axis to the remotest fiber, the intensity of the diagonal tension will be different at different points in the section, and will change with different proportionate dimensions of length to depth of beam. For the composite structure of reinforced concrete beams, an analysis of the web stresses, and particularly of the diagonal tensile stresses, is very complex; and when the variations due to a change from no horizontal tensile stress in the concrete at remotest fiber to the presence of horizontal tensile stress at some point below the neutral axis are considered, the problem becomes more complex and indefinite. Under these circumstances, in designing recourse is had to the use of the calculated vertical shearing stress as a means of comparing or measuring the diagonal tensile stresses developed, it being understood that the vertical shearing stress is not the numerical equivalent of the diagonal tensile stress, and that there is not even a constant ratio between them. It is here recommended that the maximum vertical shearing stress in a section be used as the means of comparison of the resistance to diagonal tensile stress developed in the concrete in beams not having web reinforcement.

Even after the concrete has reached its limit of resistance to diagonal tension, if the beam has web reinforcement, conditions of beam action will continue to prevail, at least through the compression area, and the web reinforcement will be called on to resist only a part of the web stresses. From experiments with beams it is concluded that it is safe practice to use only two-thirds of the external vertical shear in making calculations of the stresses that come on stirrups, diagonal web pieces, and bent-up bars, and it is here recommended for calculations in designing that two-thirds of the external vertical shear be taken as producing stresses in web reinforcement.

It is well established that vertical members attached to or looped about horizontal members, inclined members secured to horizontal members in such a way as to insure against slip, and

the bending of a part of the longitudinal reinforcement at an angle, will increase the strength of a beam against failure by diagonal tension, and that a well-designed and well-distributed web reinforcement may under the best conditions increase the total vertical shear carried to a value as much as three times that obtained when the bars are all horizontal and no web reinforcement is used.

When web reinforcement comes into action as the principal tension web resistance, the bond stresses between the longitudinal bars and the concrete are not distributed as uniformly along the bars as they otherwise would be, but tend to be concentrated at and near stirrups, and at and near the points where bars are bent up. When stirrups are not rigidly attached to the longitudinal bars, and the proportioning of bars and stirrup spacing is such that local slip of bars occur at stirrups, the effectiveness of the stirrups is impaired, though the presence of stirrups still gives an element of toughness against diagonal tension failure.

Sufficient bond resistance between the concrete and the stirrups or diagonals must be provided in the compression area of the beam.

The longitudinal spacing of vertical stirrups should not exceed one-half the depth of beam, and that of inclined members should not exceed three-fourths of the depth of beam.

Bending of longitudinal reinforcing bars at an angle across the web of the beam may be considered as adding to diagonal tension resistance for a horizontal distance from the point of bending equal to three-fourths of the depth of beam. Where the bending is made at two or more points, the distance between points of bending should not exceed three-fourths of the depth of the beam. In the case of a restrained beam the effect of bending up a bar at the bottom of the beam in resisting diagonal tension may not be taken as extending beyond a section at the point of inflection, and the effect of bending down a bar in the region of negative moment may be taken as extending from the point of bending down of bar nearest the support to a section not more than three-fourths of the depth of beam beyond the point of bending down of bar farthest from the support but not beyond the point of inflection. In case stirrups are used in the beam away from the region in which the bent bars are considered effec-

tive, a stirrup should be placed not farther than a distance equal to one-fourth the depth of beam from the limiting sections defined above. In case the web resistance required through the region of bent bars is greater than that furnished by the bent bars, sufficient additional web reinforcement in the form of stirrups or attached diagonals should be provided. The higher resistance to diagonal tension stresses given by unit frames having the stirrups and bent-up bars securely connected together both longitudinally and laterally is worthy of recognition. It is necessary that a limit be placed on the amount of shear which may be allowed in a beam; for when web reinforcement sufficiently efficient to give very high web resistance is used, at the higher stresses the concrete in the beam becomes checked and cracked in such a way as to endanger its durability as well as its strength.

The section to be taken as the critical section in the calculation of shearing stresses will generally be the one having the maximum vertical shear, though experiments show that the section at which diagonal tension failures occur is not just at a support even though the shear at the latter point be much greater.

In the case of restrained beams, the first stirrup or the point of bending down of bar should be placed not farther than one-half of the depth of beam away from the face of the support.

It is important that adequate bond strength or anchorage be provided to develop fully the assumed strength of all web reinforcement.

Low bond stresses in the longitudinal bars are helpful in giving resistance against diagonal tension failures and anchorage of longitudinal bars at the ends of the beams or in the supports is advantageous.

It should be noted that it is on the tension side of a beam that diagonal tension develops in a critical way, and that proper connection should always be made between stirrups or other web reinforcement and the longitudinal tension reinforcement, whether the latter is on the lower side of the beam or on its upper side. Where negative moment exists, as is the case near the supports in a continuous beam, web reinforcement to be effective must be looped over or wrapped around or be connected with the longitudinal tension reinforcing bars at the top of the beam in

the same way as is necessary at the bottom of the beam at sections where the bending moment is positive.

Inasmuch as the smaller the longitudinal deformations in the horizontal reinforcement are, the less the tendency for the formation of diagonal cracks, a beam will be strengthened against diagonal tension failure by so arranging and proportioning the horizontal reinforcement that the unit stresses at points of large shear shall be relatively low.

It does not seem feasible to make a complete analysis of the action of web reinforcement, and more or less empirical methods of calculation are therefore employed. Limiting values of working stresses for different types of web reinforcement are given in Chapter VIII, Section 5. The conditions apply to cases commonly met in design. It is assumed that adequate bond resistance or anchorage of all web reinforcement will be provided.

When a flat slab rests on a column, or a column bears on a footing, the vertical shearing stresses in the slab or footing immediately adjacent to the column are termed punching shearing stresses. The element of diagonal tension, being a function of the bending moment as well as of shear, may be small in such cases, or may be otherwise provided for. For this reason the permissible limit of stress for punching shear may be higher than the allowable limit when the shearing stress is used as a means of comparing diagonal tensile stress. The working values recommended are given in Chapter VIII, Section 5.

9. COLUMNS.

By columns are meant compression members of which the ratio of unsupported length to least width exceeds about four, and which are provided with reinforcement of one of the forms hereafter described.

It is recommended that the ratio of unsupported length of column to its least width be limited to 15.

The effective area of hooped columns or columns reinforced with structural shapes shall be taken as the area within the circle enclosing the spiral or the polygon enclosing the structural shapes.

Columns may be reinforced by longitudinal bars; by bands, hoops, or spirals, together with longitudinal bars; or by structural forms which are sufficiently rigid to have value in them-

selves as columns. The general effect of closely spaced hooping is to greatly increase the toughness of the column and to add to its ultimate strength, but hooping has little effect on its behavior within the limit of elasticity. It thus renders the concrete a safer and more reliable material, and should permit the use of a somewhat higher working stress. The beneficial effects of toughening are adequately provided by a moderate amount of hooping, a larger amount serving mainly to increase the ultimate strength and the deformation possible before ultimate failure.

Composite columns of structural steel and concrete in which the steel forms a column by itself should be designed with caution. To classify this type as a concrete column reinforced with structural steel is hardly permissible, as the steel generally will take the greater part of the load. When this type of column is used, the concrete should not be relied upon to tie the steel units together nor to transmit stresses from one unit to another. The units should be adequately tied together by tie plates or lattice bars, which, together with other details, such as splices, etc., should be designed in conformity with standard practice for structural steel. The concrete may exert a beneficial effect in restraining the steel from lateral deflection and also in increasing the carrying capacity of the column. The proportion of load to be carried by the concrete will depend on the form of the column and the method of construction. Generally, for high percentages of steel, the concrete will develop relatively low unit-stresses, and caution should be used in placing dependence on the concrete.

The following recommendations are made for the relative working stresses in the concrete for the several types of columns:

(*a*) Columns with longitudinal reinforcement to the extent of not less than 1 per cent and not more than 4 per cent, and with lateral ties of not less than $\frac{1}{4}$ in. in diameter 12 in. apart, nor more than 16 diameters of the longitudinal bar: the unit stress recommended for axial compression, on concrete piers having a length not more than four diameters, in Chapter VIII, Section 3.

(*b*) Columns reinforced with not less than 1 per cent and not more than 4 per cent of longitudinal bars and with circular hoops or spirals not less than 1 per cent of the volume of the concrete and as hereinafter specified: a unit stress 55 per cent higher than given for (*a*), provided the ratio of unsupported length of column to diameter of the hooped core is not more than 10.

The foregoing recommendations are based on the following conditions:

It is recommended that the minimum size of columns to which the working stresses may be applied be 12 in. out to out.

In all cases longitudinal reinforcement is assumed to carry its proportion of stress in accordance with Section 3 (*c*) 6 of this chapter. The hoops or bands are not to be counted on directly as adding to the strength of the column.

Longitudinal reinforcement bars should be maintained straight, and should have sufficient lateral support to be securely held in place until the concrete has set.

Where hooping is used, the total amount of such reinforcement shall be not less than 1 per cent of the volume of the column, enclosed. The clear spacing of such hooping shall be not greater than one-sixth the diameter of the enclosed column and preferably not greater than one-tenth, and in no case more than $2\frac{1}{2}$ in. Hooping is to be circular and the ends of bands must be united in such a way as to develop their full strength. Adequate means must be provided to hold bands or hoops in place so as to form a column, the core of which shall be straight and well centered. The strength of hooped columns depends very much upon the ratio of length to diameter of hooped core, and the strength due to hooping decreases rapidly as this ratio increases beyond five. The working stresses recommended are for hooped columns with a length of not more than ten diameters of the hooped core.

The Committee has no recommendation to make for a formula for working stresses for columns longer than ten diameters.

Bending stresses due to eccentric loads, such as unequal spans of beams, and to lateral forces, must be provided for by increasing the section until the maximum stress does not exceed the values above specified. Where tension is possible in the

longitudinal bars of the column, adequate connection between the ends of the bars must be provided to take this tension.

10. Reinforcing for shrinkage and temperature stresses.

When areas of concrete too large to expand and contract freely as a whole are exposed to atmospheric conditions, the changes of form due to shrinkage and to action of temperature are such that cracks may occur in the mass unless precautions are taken to distribute the stresses so as to prevent the cracks altogether or to render them very small. The distance apart of the cracks, and consequently their size, will be directly proportional to the diameter of the reinforcement and to the tensile strength of the concrete, and inversely proportional to the percentage of reinforcement and also to its bond resistance per unit of surface area. To be most effective, therefore, reinforcement (in amount generally not less than one-third of one per cent of the gross area) of a form which will develop a high bond resistance should be placed near the exposed surface and be well distributed. Where openings occur the area of cross-section of the reinforcement should not be reduced. The allowable size and spacing of cracks depends on various considerations, such as the necessity for water-tightness, the importance of appearance of the surface, and the atmospheric changes.

The tendency of concrete to shrink makes it necessary, except where expansion is provided for, to thoroughly connect the component parts of the frame of articulated structures, such as floor and wall members in buildings, by the use of suitable reinforcing material. The amount of reinforcement for such connection should bear some relation to the size of the members connected, larger and heavier members requiring stronger connections. The reinforcing bars should be extended beyond the critical section far enough, or should be sufficiently anchored to develop their full tensile strength.

11. Flat slab.

The continuous flat slab reinforced in two or more directions and built monolithically with the supporting columns (without beams or girders) is a type of construction which is now extensively

used and which has recognized advantages for certain types of structures as, for example, warehouses in which large, open floor space is desired. In its construction, there is excellent opportunity for inspecting the position of the reinforcement. The conditions attending depositing and placing of concrete are favorable to securing uniformity and soundness in the concrete. The recommendations in the following paragraphs relate to flat slabs extending over several rows of panels in each direction. Necessarily the treatment is more or less empirical.

The coefficients and moments given relate to uniformly distributed loads.

(*a*) *Column Capital.*—It is usual in flat slab construction to enlarge the supporting columns at their top, thus forming column capitals. The size and shape of the column capital affect the strength of the structure in several ways. The moment of the external forces which the slab is called upon to resist is dependent upon the size of the capital; the section of the slab immediately above the upper periphery of the capital carries the highest amount of punching shear; and the bending moment developed in the column by an eccentric or unbalanced loading of the slab is greatest at the under surface of the slab. Generally the horizontal section of the column capital should be round or square with rounded corners. In oblong panels the section may be oval or oblong, with dimensions proportional to the panel dimensions. For computation purposes, the diameter of the column capital will be considered to be measured where its vertical thickness is at least $1\frac{1}{2}$ in., provided the slope of the capital below this point nowhere makes an angle with the vertical of more than 45 deg. In case a cap is placed above the column capital, the part of this cap within a cone made by extending the lines of the column capital upward at the slope of 45 deg. to the bottom of the slab or dropped panel may be considered as part of the column capital in determining the diameter for design purposes. Without attempting to limit the size of the column capital for special cases, it is recommended that the diameter of the column capital (or its dimension parallel to the edge of the panel) generally be made not less than one-fifth of the dimension of the panel from center to center of adjacent columns. A diameter equal to 0.225 of the panel length has been used quite widely and

acceptably. For heavy loads or large panels especial attention should be given to designing and reinforcing the column capital with respect to compressive stresses and bending moments. In the case of heavy loads or large panels, and where the conditions of the panel loading or variations in panel length or other conditions cause high bending stresses in the column, and also for column capitals smaller than the size herein recommended, especial attention should be given to designing and reinforcing the column capital with respect to compression and to rigidity of connection to floor slab.

(b) *Dropped Panel.*—In one type of construction the slab is thickened throughout an area surrounding the column capital. The square or oblong of thickened slab thus formed is called a dropped panel or a drop. The thickness and the width of the dropped panel may be governed by the amount of resisting moment to be provided (the compressive stress in the concrete being dependent upon both thickness and width), or its thickness may be governed by the resistance to shear required at the edge of the column capital and its width by the allowable compressive stresses and shearing stresses in the thinner portion of the slab adjacent to the dropped panel. Generally, however, it is recommended that the width of the dropped panel be at least four-tenths of the corresponding side of the panel as measured from center to center of columns, and that the offset in thickness be not more than five-tenths of the thickness of the slab outside the dropped panel.

(c) *Slab Thickness.*—In the design of a slab, the resistance to bending and to shearing forces will largely govern the thickness, and, in the case of large panels with light loads, resistance to deflection may be a controlling factor. The following formulas for minimum thicknesses are recommended as general rules of design when the diameter of the column capital is not less than one-fifth of the dimension of the panel from center to center of adjacent columns, the larger dimension being used in the case of oblong panels. For notation, let

t = total thickness of slab in inches.
L = panel length in feet.
w = sum of live load and dead load in pounds per square foot.

Then, for a slab without dropped panels, minimum $t=0.024\ L\sqrt{w}+1\frac{1}{2}$; for a slab with dropped panels, minimum $t=0.02\ L\sqrt{w}+1$; for a dropped panel whose width is four-tenths of the panel length, minimum $t=0.03\ L\sqrt{w}+1\frac{1}{2}$.

In no case should the slab thickness be made less than six inches, nor should the thickness of a floor slab be made less than one-thirty-second of the panel length, nor the thickness of a roof slab less than one-fortieth of the panel length.

(*d*) *Bending and Resisting Moments in Slabs.*—If a vertical section of a slab be taken across a panel along a line midway

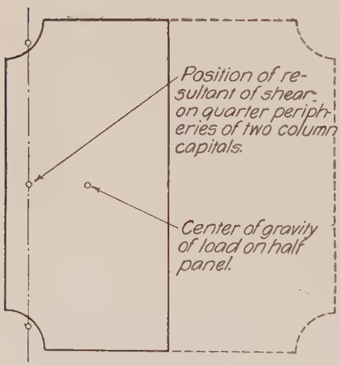

FIG. 1.

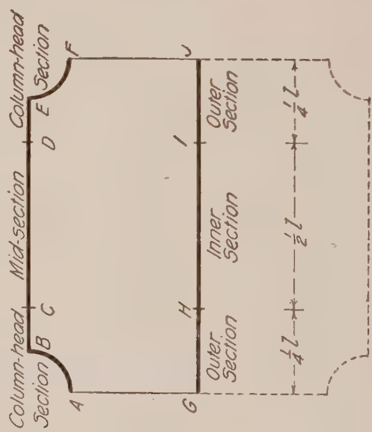

FIG. 2.

between columns, and if another section be taken along an edge of the panel parallel to the first section, but skirting the part of the periphery of the column capitals at the two corners of the panels, the moment of the couple formed by the external load on the half panel, exclusive of that over the column capital (sum of dead and live load) and the resultant of the external shear or reaction at the support at the two column capitals (see Fig. 1), may be found by ordinary static analysis. It will be noted that the edges of the area here considered are along lines of zero shear except around the column capitals. This moment of the external forces acting on the half panel will be resisted by the numerical sum of (*a*) the moment of the internal stresses at the section of the panel midway between columns (positive resisting

moment) and (*b*) the moment of the internal stresses at the section referred to at the end of the panel (negative resisting moment). In the curved portion of the end section (that skirting the column), the stresses considered are the components which act parallel to the normal stresses on the straight portion of the section. Analysis shows that, for a uniformly distributed load, and round columns, and square panels, the numerical sum of the positive moment and the negative moment at the two sections named is given quite closely by the equation

$$M_x = \frac{1}{8} wl \left(l - \frac{2}{3} c\right)^2.$$

In this formula and in those which follow relating to oblong panels,

w = sum of the live and dead load per unit of area;
l = side of a square panel measured from center to center of columns;
l_1 = one side of the oblong panel measured from center to center of columns;
l_2 = other side of oblong panel measured in the same way;
c = diameter of the column capital;
M_x = numerical sum of positive moment and negative moment in one direction.
M_y = numerical sum of positive moment and negative moment in the other direction.

(See paper and closure, Statical Limitations upon the Steel Requirement in Reinforced Concrete Flat Slab Floors, by John R. Nichols, Jun. Am. Soc. C. E., Transactions Am. Soc. C. E., Vol. LXXVII.)

For oblong panels, the equations for the numerical sums of the positive moment and the negative moment at the two sections named become,

$$M_x = \frac{1}{8} wl_2 \left(l_1 - \frac{2}{3} c\right)^2$$
$$M_y = \frac{1}{8} wl_1 \left(l_2 - \frac{2}{3} c\right)^2$$

where M_x is the numerical sum of the positive moment and the negative moment for the sections parallel to the dimension l_2, and M_y is the numerical sum of the positive moment and the negative moment for the sections parallel to the dimension l_1.

What proportion of the total resistance exists as positive moment and what as negative moment is not readily determined. The amount of the positive moment and that of the negative moment may be expected to vary somewhat with the design of the slab. It seems proper, however, to make the division of total resisting moment in the ratio of three-eighths for the positive moment to five-eighths for the negative moment.

With reference to variations in stress along the sections, it is evident from conditions of flexure that the resisting moment is not distributed uniformly along either the section of positive moment or that of negative moment. As the law of the distribution is not known definitely, it will be necessary to make an empirical apportionment along the sections; and it will be considered sufficiently accurate generally to divide the sections into two parts and to use an average value over each part of the panel section.

The relatively large breadth of structure in a flat slab makes the effect of local variations in the concrete less than would be the case for narrow members like beams. The tensile resistance of the concrete is less affected by cracks. Measurements of deformations in buildings under heavy load indicate the presence of considerable tensile resistance in the concrete, and the presence of this tensile resistance acts to decrease the intensity of the compressive stresses. It is believed that the use of moment coefficients somewhat less than those given in a preceding paragraph as derived by analysis is warranted, the calculations of resisting moment and stresses in concrete and reinforcement being made according to the assumptions specified in this report and no change being made in the values of the working stresses ordinarily used. Accordingly, the values of the moments which are recommended for use are somewhat less than those derived by analysis. The values given may be used when the column capitals are round, oval, square, or oblong.

(*e*) *Names for Moment Sections.*—For convenience, that portion of the section across a panel along a line midway between columns which lies within the middle two quarters of the width of the panel (HI, Fig. 2) will be called the inner section, and that portion in the two outer quarters of the width of the panel

(GH and IJ, Fig. 2) will be called the outer sections. Of the section which follows a panel edge from column capital to column capital and which includes the quarter peripheries of the edges of two column capitals, that portion within the middle two quarters of the panel width (CD, Fig. 2) will be called the mid-section, and the two remaining portions (ABC and DEF, Fig. 2), each having a projected width equal to one-fourth of the panel width, will be called the column-head sections.

(*f*) *Positive Moment.*—For a square interior panel, it is recommended that the positive moment for a section in the middle of a panel extending across its width be taken as $\frac{1}{25} wl\ (l - \frac{2}{3}c)^2$. Of this moment, at least 25 per cent should be provided for in the inner section; in the two outer sections of the panel at least 55 per cent of the specified moment should be provided for in slabs not having dropped panels, and at least 60 per cent in slabs having dropped panels, except that in calculations to determine necessary thickness of slab away from the dropped panel at least 70 per cent of the positive moment should be considered as acting in the two outer sections.

(*g*) *Negative Moment.*—For a square interior panel, it is recommended that the negative moment for a section which follows a panel edge from column capital to column capital and which includes the quarter peripheries of the edges of the two column capitals (the section altogether forming the projected width of the panel) be taken as $\frac{1}{15}\ wl\ (l - \frac{2}{3}c)^2$. Of this negative moment, at least 20 per cent should be provided for in the mid-section and at least 65 per cent in the two column-head sections of the panel, except that in slabs having dropped panels at least 80 per cent of the specified negative moment should be provided for in the two column-head sections of the panel.

(*h*) *Moments for Oblong Panels.*—When the length of a panel does not exceed the breadth by more than 5 per cent, computation may be made on the basis of a square panel with sides equal to the mean of the length and the breadth.

When the long side of an interior oblong panel exceeds the short side by more than one-twentieth and by not more than one-third of the short side, it is recommended that the positive

moment be taken as $\frac{1}{25} wl_2 (l_1 - \frac{2}{3} c)^2$ on a section parallel to the dimension l_2, and $\frac{1}{25} wl_1 (l_2 - \frac{2}{3} c)^2$ on a section parallel to the dimension l_1; and that the negative moment be taken as $\frac{1}{15} wl_2 (l_1 - \frac{2}{3} c)^2$ on a section at the edge of the panel corresponding to the dimension l_2, and $\frac{1}{15} wl_1 (l_2 - \frac{2}{3} c)^2$ at a section in the other direction. The limitations of the apportionment of moment between inner section and outer section and between mid-section and column-head sections may be the same as for square panels.

(*i*) *Wall Panels.*—The coefficient of negative moment at the first row of columns away from the wall should be increased 20 per cent over that required for interior panels, and likewise the coefficient of positive moment at the section half way to the wall should be increased by 20 per cent. If girders are not provided along the wall or the slab does not project as a cantilever beyond the column line, the reinforcement parallel to the wall for the negative moment in the column-head section and for the positive moment in the outer section should be increased by 20 per cent. If the wall is carried by the slab this concentrated load should be provided for in the design of the slab. The coefficient of negative moments at the wall to take bending in the direction perpendicular to the wall line may be determined by the conditions of restraint and fixedness as found from the relative stiffness of columns and slab, but in no case should it be taken as less than one-half of that for interior panels.

(*j*) *Reinforcement.*—In the calculation of moments all the reinforcing bars which cross the section under consideration and which fulfill the requirements given under Paragraph (*l*) of this chapter may be used. For a column-head section reinforcing bars parallel to the straight portion of the section do not contribute to the negative resisting moment for the column-head section in question. In the case of four-way reinforcement the sectional area of the diagonal bars multiplied by the sine of the angle between the diagonal of the panel and the straight portion of the section under consideration may be taken to act as reinforcement in a rectangular direction.

(*k*) *Point of Inflection.*—For the purpose of making calculations of moments at sections away from the sections of negative moment and positive moment already specified, the point of inflection on any line parallel to a panel edge may be taken as one-fifth of the clear distance on that line between the two sections of negative moment at the opposite ends of the panel indicated in Paragraph (*e*), of this chapter. For slabs having dropped panels the coefficient of one-fourth should be used instead of one-fifth.

(*l*) *Arrangement of Reinforcement.*—The design should include adequate provision for securing the reinforcement in place so as to take not only the maximum moments but the moments at intermediate sections. All bars in rectangular bands or diagonal bands should extend on each side of a section of maximum moment, either positive or negative, to points at least twenty diameters beyond the point of inflection as defined herein or be hooked or anchored at the point of inflection. In addition to this provision bars in diagonal bands used as reinforcement for negative moment should extend on each side of a line drawn through the column center at right angles to the direction of the band at least a distance equal to thirty-five one-hundredths of the panel length, and bars in diagonal bands used as reinforcement for positive moment should extend on each side of a diagonal through the center of the panel at least a distance equal to thirty-five one-hundredths of the panel length; and no splice by lapping should be permitted at or near regions of maximum stress except as just described. Continuity of reinforcing bars is considered to have advantages, and it is recommended that not more than one-third of the reinforcing bars in any direction be made of a length less than the distance center to center of columns in that direction. Continuous bars should not all be bent up at the same point of their length, but the zone in which this bending occurs should extend on each side of the assumed point of inflection, and should cover a width of at least one-fifteenth of the panel length. Mere draping of the bars should not be permitted. In four-way reinforcement the position of the bars in both diagonal and rectangular directions may be considered in determining whether the width of zone of bending is sufficient.

(*m*) *Reinforcement at Construction Joints.*—It is recommended that at construction joints extra reinforcing bars equal in section

to 20 per cent of the amount necessary to meet the requirements for moments at the section where the joint is made be added to the reinforcement, these bars to extend not less than 50 diameters beyond the joint on each side.

(*n*) *Tensile and Compressive Stresses.*—The usual method of calculating the tensile and compressive stresses in the concrete and in the reinforcement, based on the assumptions for internal stresses given in this chapter, should be followed. In the case of the dropped panel the section of the slab and dropped panel may be considered to act integrally for a width equal to the width of the column-head section.

(*o*) *Provision for Diagonal Tension and Shear.*—In calculations for the shearing stress which is to be used as the means of measuring the resistance to diagonal tension stress, it is recommended that the total vertical shear on two column-head sections constituting a width equal to one-half the lateral dimension of the panel, for use in the formula for determining critical shearing stresses, be considered to be one-fourth of the total dead and live load on a panel for a slab of uniform thickness, and to be three-tenths of the sum of the dead and live loads on a panel for a slab with dropped panels. The formula for shearing unit stress given in Chapter X of this report may then be written $v = \frac{0.25\,W}{bjd}$ for slabs of uniform thickness, and $v = \frac{0.30\,W}{bjd}$ for slabs with dropped panels, where W is the sum of the dead and live load on a panel, b is half the lateral dimension of the panel measured from center to center of columns, and jd is the lever arm of the resisting couple at the section.

The calculation of what is commonly called punching shear may be made on the assumption of a uniform distribution over the section of the slab around the periphery of the column capital and also of a uniform distribution over the section of the slab around the periphery of the dropped panel, using in each case an amount of vertical shear greater by 25 per cent than the total vertical shear on the section under consideration.

The values of working stresses should be those recommended for diagonal tension and shear in Chapter VIII, Section 5.

(*p*) *Walls and Openings.*—Girders or beams should be constructed to carry walls and other concentrated loads which are in

excess of the working capacity of the slab. Beams should also be provided in case openings in the floor reduce the working strength of the slab below the required carrying capacity.

(*q*) *Unusual Panels.*—The coefficients, apportionments, and thicknesses recommended are for slabs which have several rows of panels in each direction, and in which the size of the panels is approximately the same. For structures having a width of one, two, or three panels, and also for slabs having panels of markedly different sizes, an analysis should be made of the moments developed in both slab and columns, and the values given herein modified accordingly. Slabs with paneled ceiling or with depressed paneling in the floor are to be considered as coming under the recommendations herein given.

(*r*) *Bending Moments in Columns.*—Provision should be made in both wall columns and interior columns for the bending moment which will be developed by unequally loaded panels, eccentric loading, or uneven spacing of columns. The amount of moment to be taken by a column will depend upon the relative stiffness of columns and slab, and computations may be made by rational methods, such as the principle of least work, or of slope and deflection. Generally, the larger part of the unequalized negative moment will be transmitted to the columns, and the column should be designed to resist this bending moment. Especial attention should be given to wall columns and corner columns.

CHAPTER VIII.

WORKING STRESSES.

1. GENERAL ASSUMPTIONS.

The following working stresses are recommended for static loads. Proper allowances for vibration and impact are to be added to live loads where necessary to produce an equivalent static load before applying the unit stresses in proportioning parts.

In selecting the permissible working stress on concrete, the designer should be guided by the working stresses usually allowed for other materials of construction, so that all structures of the same class composed of different materials may have approximately the same degree of safety.

The following recommendations as to allowable stresses are given in the form of percentages of the ultimate strength of the particular concrete which is to be used; this ultimate strength is that developed at an age of 28 days, in cylinders 8 in. in diameter and 16 in. long, of the consistency described in Chapter IV, Section 2 (*d*), made and stored under laboratory conditions. In the absence of definite knowledge in advance of construction as to just what strength may be expected, the Committee submits the following values as those which should be obtained with materials and workmanship in accordance with the recommendations of this report.

Although occasional tests may show higher results than those here given, the Committee recommends that these values should be the maximum used in design.

TABLE OF COMPRESSIVE STRENGTHS OF DIFFERENT MIXTURES OF CONCRETE.
(In Pounds per Square Inch.)

Aggregate	1 : 3*	1 : 4½*	1 : 6*	1 : 7½*	1 : 9*
Granite, trap rock................	3300	2800	2200	1800	1400
Gravel, hard limestone and hard sandstone.....................	3000	2500	2000	1600	1300
Soft limestone and sandstone......	2200	1800	1500	1200	1000
Cinders........................	800	700	600	500	400

NOTE.—For variations in the moduli of elasticity see Chapter VIII, Section 8.

2. BEARING.

When compression is applied to a surface of concrete of at least twice the loaded area, a stress of 35 per cent of the compressive strength may be allowed in the area actually under load.

3. AXIAL COMPRESSION.

For concentric compression on a plain concrete pier, the length of which does not exceed 4 diameters, or on a column reinforced with longitudinal bars only, the length of which does not exceed 12 diameters, 22.5 per cent of the compressive strength may be allowed.

For other forms of columns the stresses obtained from the ratios given in Chapter VII, Section 9, may govern.

* Combined volume fine and coarse aggregates measured separately.

4. COMPRESSION IN EXTREME FIBER.

The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity for concrete under working stresses may be allowed to reach 32.5 per cent of the compressive strength. Adjacent to the support of continuous beams stresses 15 per cent higher may be used.

5. SHEAR AND DIAGONAL TENSION.

In calculations on beams in which the maximum shearing stress in a section is used as the means of measuring the resistance to diagonal tension stress, the following allowable values for the maximum vertical shearing stress in concrete, calculated by the method given in Chapter X, Formula 22, are recommended:

(*a*) For beams with horizontal bars only and without web reinforcement, 2 per cent of the compressive strength.

(*b*) For beams with web reinforcement consisting of vertical stirrups looped about the longitudinal reinforcing bars in the tension side of the beam and spaced horizontally not more than one-half the depth of the beam; or for beams in which longitudinal bars are bent up at an angle of not more than 45 deg. or less than 20 deg. with the axis of the beam, and the points of bending are spaced horizontally not more than three-quarters of the depth of the beam apart, not to exceed $4\frac{1}{2}$ per cent of the compressive strength.

(*c*) For a combination of bent bars and vertical stirrups looped about the reinforcing bars in the tension side of the beam and spaced horizontally not more than one-half of the depth of the beam, 5 per cent of the compressive strength.

(*d*) For beams with web reinforcement (either vertical or inclined) securely attached to the longitudinal bars in the tension side of the beam in such a way as to prevent slipping of bar past the stirrup, and spaced horizontally not more than one-half of the depth of the beam in case of vertical stirrups and not more than three-fourths of the depth of the beam in the case of inclined members, either with longitudinal bars bent up or not, 6 per cent of the compressive strength.

The web reinforcement in case any is used should be proportioned by using two-thirds of the external vertical shear in

Formula 24 or 25 in Chapter X. The effect of longitudinal bars bent up at an angle of from 20 to 45 deg. with the axis of the beam may be taken at sections of the beam in which the bent up bars contribute to diagonal tension resistance as defined under Chapter VII, Section 8, as reducing the shearing stresses to be otherwise provided for. The amount of reduction of the shearing stress by means of bent up bars will depend upon their capacity, but in no case should be taken as greater than 4½ per cent of the compressive strength of the concrete over the effective cross-section of the beam (Formula 22). The limit of tensile stress in the bent up portion of the bar calculated by Formula 25, using in this formula an amount of total shear corresponding to the reduction in shearing stress assumed for the bent up bars, may be taken as specified for the working stress of steel, but in the calculations the stress in the bar due to its part as longitudinal reinforcement of the beam should be considered. The stresses in stirrups and inclined members when combined with bent up bars are to be determined by finding the amount of the total shear which may be allowed by reason of the bent up bars, and subtracting this shear from the total external vertical shear. Two-thirds of the remainder will be the shear to be carried by the stirrups, using Formulas 24 or 25 in Chapter X.

Where punching shear occurs, provided the diagonal tension requirements are met, a shearing stress of 6 per cent of the compressive strength may be allowed.

6. BOND.

The bond stress between concrete and plain reinforcing bars may be assumed at 4 per cent of the compressive strength, or 2 per cent in the case of drawn wire. In the best types of deformed bar the bond stress may be increased, but not to exceed 5 per cent of the compressive strength of the concrete.

7. REINFORCEMENT.

The tensile or compressive stress in steel should not exceed 16,000 lb. per sq. in.

In structural steel members the working stresses adopted by the American Railway Engineering Association are recommended.

8. MODULUS OF ELASTICITY.

The value of the modulus of elasticity of concrete has a wide range, depending on the materials used, the age, the range of stresses between which it is considered, as well as other conditions. It is recommended that in computations for the position of the neutral axis, and for the resisting moment of beams and for compression of concrete in columns, it be assumed as:

(*a*) One-fortieth that of steel, when the strength of the concrete is taken as not more than 800 lb. per sq. in.
(*b*) One-fifteenth that of steel, when the strength of the concrete is taken as greater than 800 lb. per sq. in. and less than 2200 lb. per sq. in.
(*c*) One-twelfth that of steel, when the strength of the concrete is taken as greater than 2200 lb. per sq. in. and less than 2900 lb. per sq. in., and
(*d*) One-tenth that of steel, when the strength of the concrete is taken as greater than 2900 lb. per sq. in.

Although not rigorously accurate, these assumptions will give safe results. For the deflection of beams which are free to move longitudinally at the supports, in using formulas for deflection which do not take into account the tensile strength developed in the concrete, a modulus of one-eighth of that of steel is recommended.

CHAPTER IX.

CONCLUSION.

In the preparation of this Final Report, 21 members have taken a more or less active part; all members have agreed to it in its present form.

The Joint Committee acknowledges its indebtedness to its sub-committee on design, Professors Talbot, Hatt and Turneaure, for their invaluable and devoted service.

The Joint Committee believes that there is a great advantage in the co-operation of the representatives of different technical societies, and trusts that a similar combination of effort may be possible, some time in the future, to review the work done by the

present Committee, and to embody the additional knowledge which will certainly be obtained from further experimentation and practical experience with this important material of construction.

Respectfully submitted,

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ROBERT E. GRIFFITH,
SPENCER B. NEWBERRY,
EDWARD GODFREY,[1]
EGBERT J. MOORE,
LEONARD C. WASON.

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[1] Mr. Godfrey dissents from the Report in the whole matter of stirrups and their treatment. He would give stirrups and short shear members no recognition, for the reason that he holds that they have not shown themselves to have any definite value in tests and that analysis fails to show that any definite value can be ascribed to them; he also believes that dependence on stirrups to take end shear has resulted in much unsafe construction and some failures. He would take care of diagonal tension by bending up some of the main reinforcing rods and anchoring them for their full tensile strength beyond the edge of support. He recommends that bends be made close to the supports for the upper bends and at quarter points for the lower bends in beams carrying uniform load. For girders carrying beams bends should be made under the beams. For anchorage he recommends that the rod should extend 40 to 50 diameters beyond the point where it intersects a line drawn, at 45 deg. with the horizontal from the bottom of the beam at the face of the support.

He recommends that the stress in bent-up rods be assumed to be that obtained by multiplying the excess of shear over that taken by the concrete (at 40 or 50 lb. per sq. in.) by the secant of the inclination of the rod with the vertical.

Mr. Godfrey also dissents from all parts of the Report relating to rodded columns, or columns having longitudinal rods without close-spaced hooping, for the reason that he holds that such reinforcement has not shown itself to have any definite value in tests on columns, and that analysis fails to show that any definite value can be ascribed to it, when such analysis takes into account the necessity for toughness in all columns; he also believes that dependence on such reinforcement has led to much unsafe construction and many failures. He would recognize as reinforced concrete columns only such columns as have in addition to the longitudinal rods a complete system of close-spaced hooping. He objects to the reading of Chap. VII, Sec. 9, paragraph (*b*) as being capable of interpretation that hooped columns are given an advantage in the matter of unit stresses only below ten diameters in height. He recommends the standardization of hooped columns and suggests that columns be reinforced by

CHAPTER X.

APPENDIX.

SUGGESTED FORMULAS FOR REINFORCED CONCRETE CONSTRUCTION.

These formulas are based on the assumptions and principles given in the chapter on design.

1. STANDARD NOTATION.

(*a*) *Rectangular Beams.*

The following notation is recommended:

f_s = tensile unit stress in steel;
f_c = compressive unit stress in concrete;
E_s = modulus of elasticity of steel;
E_c = modulus of elasticity of concrete;
$n = \dfrac{E_s}{E_c}$;
M = moment of resistance, or bending moment in general;
A_s = steel area;
b = breadth of beam;
d = depth of beam to center of steel;
k = ratio of depth of neutral axis to depth, d;
z = depth below top to resultant of the compressive stresses;
j = ratio of lever arm of resisting couple to depth, d;
$jd = d-z$ = arm of resisting couple;
p = steel ratio $= \dfrac{A_s}{bd}$.

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a coil or hoops of round steel having a diameter one-fortieth of that of the external diameter of the column and eight upright rods wired to the same, the pitch of the coil being one-eighth of the column diameter. He would consider available for resisting compressive stress, the entire area of the concrete of a circular column or of an octagonal column, but no part of the longitudinal rods or hooping. In a square column only 83 per cent of the area of concrete would be considered available. The compression he would recommend on columns (for 2000-lb. concrete) would be:

$$P = 670 - 12\, l/d.$$

where P = allowable compression in pounds per square inch.
l = length of column in inches.
d = diameter of column in inches.

(*b*) *T-Beams.*

b = width of flange;
b' = width of stem;
t = thickness of flange.

(*c*) *Beams Reinforced for Compression.*

A' = area of compressive steel;
p' = steel ratio for compressive steel;
f_s' = compressive unit stress in steel;
C = total compressive stress in concrete;
C' = total compressive stress in steel;
d' = depth to center of compressive steel;
z = depth to resultant of C and C'.

(*d*) *Shear, Bond and Web Reinforcement.*

V = total shear;
V' = total shear producing stress in reinforcement;
v = shearing unit stress;
u = bond stress per unit area of bar;
o = circumference or perimeter of bar;
Σo = sum of the perimeters of all bars;
T = total stress in single reinforcing member;
s = horizontal spacing of reinforcing members.

(*e*) *Columns.*

A = total net area;
A_s = area of longitudinal steel;
A_c = area of concrete;
P = total safe load.

2. FORMULAS.

(*a*) *Rectangular Beams.*

Position of neutral axis,

$$k=\sqrt{2pn+(pn)^2}-pn \quad \text{(1)}$$

Arm of resisting couple,

$$j = 1 - \frac{1}{3}k \quad \ldots\ldots (2)$$

[For $f_s = 15000$ to 16000 and $f_c = 600$ to 650, j may be taken at $\frac{7}{8}$.]

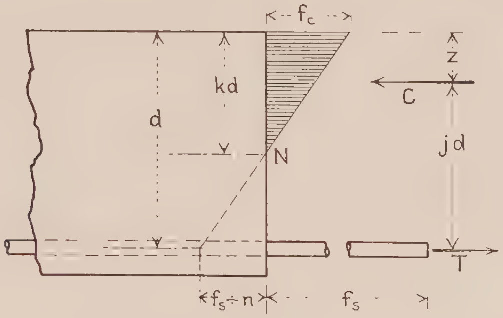

FIG. 1.

Fiber stresses,

$$f_s = \frac{M}{A_s jd} = \frac{M}{pjbd^2} \quad \ldots\ldots (3)$$

$$f_c = \frac{2M}{jkbd^2} = \frac{2pf_s}{k} \quad \ldots\ldots (4)$$

Steel ratio, for balanced reinforcement,

$$p = \tfrac{1}{2} \cdot \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)} \quad \ldots\ldots (5)$$

(*b*) *T-Beams.*

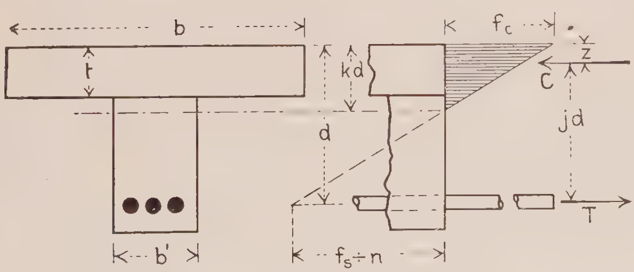

FIG. 2.

Case I. When the neutral axis lies in the flange, use the formulas for rectangular beams.

Case II. When the neutral axis lies in the stem.

The following formulas neglect the compression in the stem.

Position of neutral axis,

$$kd = \frac{2ndA_s + bt^2}{2nA_s + 2bt} \quad \text{.......................(6)}$$

Position of resultant compression,

$$z = \frac{3kd - 2t}{2kd - t} \cdot \frac{t}{3} \quad \text{.......................(7)}$$

Arm of resisting couple,

$$jd = d - z \quad \text{.......................(8)}$$

Fiber stresses,

$$f_s = \frac{M}{A_s jd} \quad \text{.......................(9)}$$

$$f_c = \frac{Mkd}{bt(kd - \frac{1}{2}t)\,jd} = \frac{f_s}{n} \cdot \frac{k}{1-k} \quad \text{.................(10)}$$

(For approximate results the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd = \sqrt{\frac{2ndA_s + (b-b')t^2}{b'} + \left(\frac{nA_s + (b-b')t}{b'}\right)^2} - \frac{nA_s + (b-b')t}{b'} \quad \text{.....(11)}$$

Position of resultant compression,

$$z = \frac{(kdt^2 - \frac{2}{3}t^3)b + [(kd-t)^2\,(t + \frac{1}{3}(kd-t)\,)]b'}{t(2kd-t)b + (kd-t)^2 b'} \quad \text{.............(12)}$$

Arm of resisting couple,

$$jd = d - z \quad \text{.......................(13)}$$

Fiber stresses,

$$f_s = \frac{M}{A_s jd} \quad \text{.......................(14)}$$

$$f_c = \frac{2Mkd}{[(2kd-t)bt + (kd-t)^2 b']jd} \quad \text{.......................(15)}$$

(*c*) *Beams Reinforced for Compression.*

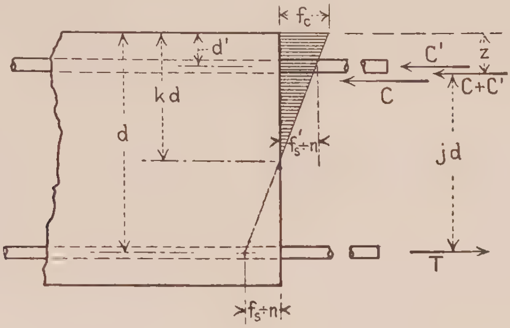

FIG. 3.

Position of neutral axis,

$$k=\sqrt{2n\left(p+p'\frac{d'}{d}\right)+n^2(p+p')^2}-n(p+p') \quad \ldots\ldots\ldots\ldots (16)$$

Position of resultant compression,

$$z=\frac{\frac{1}{3}k^3d+2p'nd'\left(k-\frac{d'}{d}\right)}{k^2+2p'n\left(k-\frac{d'}{d}\right)} \quad \ldots\ldots\ldots\ldots (17)$$

Arm of resisting couple,

$$jd=d-z \quad \ldots\ldots\ldots\ldots (18)$$

Fiber stresses,

$$f_c=\frac{6M}{bd^2\left[3k-k^2+\frac{6p'n}{k}\left(k-\frac{d'}{d}\right)\left(1-\frac{d'}{d}\right)\right]} \quad \ldots\ldots\ldots\ldots (19)$$

$$f_s=\frac{M}{pjbd^2}=nf_c\frac{1-k}{k} \quad \ldots\ldots\ldots\ldots (20)$$

$$f_s'=nf_c\frac{k-\frac{d'}{d}}{k} \quad \ldots\ldots\ldots\ldots (21)$$

(*d*) *Shear, Bond, and Web Reinforcement.*

For rectangular beams,

$$v=\frac{V}{bjd} \quad \ldots\ldots\ldots\ldots (22)$$

$$u=\frac{V}{jd\,.\,\Sigma o} \quad \ldots\ldots\ldots\ldots (23)$$

[For approximate results j may be taken at $\frac{7}{8}$.]

The stresses in web reinforcement may be estimated by means of the following formulas:

Vertical web reinforcement,

$$T=\frac{V's}{jd} \qquad (24)$$

Bars bent up at angles between 20 and 45 deg. with the horizontal and web members inclined at 45 deg.,

$$T=\tfrac{3}{4}\frac{V's}{jd} \qquad (25)$$

In the text of the report it is recommended that two-thirds of the external vertical shear (total shear) at any section be taken as the amount of total shear producing stress in the web reinforcement. V' therefore equals two-thirds of V.

The same formulas apply to beams reinforced for compression as regards shear and bond stress for tensile steel.

For T-Beams,

$$v=\frac{V}{b'jd} \qquad (26)$$

$$u=\frac{V}{jd\ .\ \Sigma o} \qquad (27)$$

[For approximate results j may be taken at $\frac{7}{8}$.]

(e) *Columns.*

Total safe load,

$$P=f_c(A_c+nA_s)=f_cA(1+(n-1)p) \qquad (28)$$

Unit stresses,

$$f_c=\frac{P}{A(1+(n-1)p)} \qquad (29)$$

$$f_s=nf_c \qquad (30)$$

ANNUAL REPORT OF BOARD OF DIRECTION.

American Concrete Institute.

During the first half of the last fiscal year headquarters were maintained in Philadelphia with Mr. C. L. Fish in charge as secretary, succeeded later by Mr. J. M. Goodell, as acting secretary, and under his management several issues of the Journal and the program for the last convention was arranged and carried out. His active connection with the Institute ceased at the close of the 1916 convention, although he has assisted the present secretary in editing Vol. XII of Proceedings which included the papers presented at the convention of Feb. 14–17, 1916.

Immediately after the adjournment of the last convention the Philadelphia headquarters were discontinued and all the file and a limited number of the old publications were sent to Boston, where headquarters have since been maintained at 27 School St.

The surplus publications that had for a number of years been kept in a storage warehouse, and for which there has been but little call, were given to the Free Public Library of the city of Philadelphia, Thirteenth and Locust Sts., who agreed to care for and distribute them for a reasonable consideration. All future inquiries for Vol. VIII or older publications should be sent direct to the library.

After many delays Vol. XII was issued the middle of last December; Vol. IX is just going through the press and will be distributed soon. Vol. X, which completes all of the arrears of our publications, except a few papers of Vol. XI, is now in the printers hands and it is hoped it will be distributed during March.

Fourteen technical committees have been busy during the year.

Our finances, by strict economy, are able to meet our necessary expenses. For details see the Treasurer's report. The balance on hand Feb. 1, 1917, was $3922.16. Vol. IX and X will reduce this by about $1800.

Our membership is not as large as it ought to be in order to make the Institute strong as an educational influence in the concrete field or to produce sufficient revenue to enable your Board of Direction to do as much for the members as they would like. Every member is urged to aid the systematic effort that is now being made to increase the membership by doing each small task that is asked of you, promptly.

Your board feel that the need and usefulness of the Institute has been so well demonstrated that the industry must and will aid its further growth. Under the new Board of Direction and officers, we are sure that the Institute will continue to grow in usefulness and influence.

Respectfully submitted,

The Board of Direction.

Leonard C. Wason,
President.

ANNUAL REPORT OF TREASURER.

At the annual convention the Treasurer submitted the reports of the auditors, Cooley & Marvin Co., Accountants and Engineers, of Boston, Mass.; first, for the twelve months ending June 30, 1916, and second, for the seven months ending January 31, 1917. These reports are as follows:

BOSTON, MASS., Feb. 2, 1917.

The American Concrete Institute,
27 School St.
Boston, Mass.

DEAR SIRS:

In accordance with your instructions we have made an examination of the books and records of the American Concrete Institute for the twelve months ending June 30, 1916, for the purpose of verifying the cash transactions of the period and presenting the financial condition of the American Concrete Institute at that date.

We submit herewith two exhibits, as follows:

Exhibit A. Statement of Condition as at June 30, 1916.
Exhibit B. Statement of Receipts and Disbursements for the year ended June 30, 1916.

The cash in bank, amounting to $2,015.35, as shown by the cash book and ledger was verified by reconciliation with the statement rendered by the depositary as at Dec. 31, 1916, accounting for all transactions between that date and June 30, 1916. During our examination paid checks, all of which were properly approved by your President and Secretary or Acting Secretary, were seen for all disbursements.

In support of the inventory as shown on the accompanying statement of condition, we have examined the inventory records in detail and have found the total to be in agreement with the amount shown. The remaining items on the statement of condition are shown in accordance with the books, and have not been further verified by us.

We have not attempted to determine the income which should have been derived during the year, and have restricted our examination to accounting for the disposition of all cash shown to have been received.

We hereby certify:

1. That all cash shown to have been received has been accounted for, and that we have seen satisfactory evidence of payment for all disbursements.
2. That the cash in bank, amounting to $2,015.35, at June 30, 1916, was on hand at that date.
3. That the Statement of Condition (Exhibit A) is in accordance with the books and, subject to the foregoing comments, in our opinion properly presents the financial condition of the American Concrete Institute at June 30, 1916.

Yours very truly,

COOLEY & MARVIN CO.

EXHIBIT A.

STATEMENT OF CONDITION.

As at June 30, 1916.

ASSETS.

Cash in bank		$2,015.35
Accounts receivable:		
Dues	$45.00	
Subscriptions	50.00	
		95.00
Inventories:		
Journals	$431.10	
Proceedings	119.10	
Supplies	67.75	
Membership pins	22.50	
Preprints, Standards, etc	7.43	
		647.88
		$2,758.23

LIABILITIES.

Advances from members and non-members for bindings, Proceedings, dues, etc	$123.89
Surplus	2,634.34
	$2,758.23

EXHIBIT B.

STATEMENT OF RECEIPTS AND DISBURSEMENTS

For the Year ended June 30, 1916.

RECEIPTS.

Balance July 1, 1915			$1,896.75
Dues		$3,978.00	
Subscriptions		4,900.00	
Publications:			
Sales Proceedings	$225.50		
" Standards	1,199.74		
" Journals	52.47		
" Membership certificates	9.50		
		1,487.21	
Cement Products Exhibition Co		500.00	
Sale of reprints, refund on rent, etc		101.55	
Sale of office furniture		60.00	
Interest on bank deposit		35.19	
Miscellaneous		34.86	
			11,096.81
Total receipts			$12,993.56

DISBURSEMENTS.

Printing, etc.:			
Proceedings	$171.05		
Journals	4,340.35		
Standards, Reports, etc	160.00		
		$4,671.40	
Salaries		2,200.40	
R. L. Humphrey		1,147.79	
Convention expenses		1,097.13	
Office and miscellaneous expenses		574.27	
Auditing		376.60	
Office rent		333.36	
Membership campaign		286.78	
Edison fire		156.10	
Storage and moving		132.04	
Membership buttons and certificates		2.34	
Total disbursements			10,978.21
Balance June 30, 1916—Exhibit A			$2,015.35

Boston, Mass., Feb. 3, 1917.

The American Concrete Institute,
27 School Street,
Boston, Mass.

Dear Sirs:

In accordance with your instructions we have made an examination of the books and records of the American Concrete Institute for the seven months ended Jan. 31, 1917, for the purpose of verifying the cash transactions of the period and presenting the financial condition of the American Concrete Institute at that date.

We submit herewith two exhibits, as follows:

Exhibit A. Statement of Condition as at Jan. 31, 1917.
Exhibit B. Statement of Receipts and Disbursements for the seven months ended Jan. 31, 1917.

The cash in bank, amounting to $3,922.16, as shown by the cash book and ledger was verified by reconciliation with the statement rendered by the depositary as at Jan. 31, 1917. During our examination paid checks, all of which were properly approved by your President and Secretary or Acting Secretary, were seen for all disbursements.

The remaining items on the statement of condition are shown in accordance with the books, and have not been further verified by us.

We have not attempted to determine the income which should have been derived during the year, and have restricted our examination to accounting for the disposition of all cash shown to have been received.

We hereby certify:

1. That all cash shown to have been received has been accounted for, and that we have seen satisfactory evidence of payment for all disbursements.
2. That the cash in bank, amounting to $3,922.16 at Jan. 31, 1917, was on hand at that date.
3. That the Statement of Condition (Exhibit A) is in accordance with the books and, subject to the foregoing comments, in our opinion properly presents the financial condition of the American Concrete Institute at Jan. 31, 1917.

Yours very truly,

Cooley & Marvin Co.

EXHIBIT A.

Statement of Condition

As at Jan. 31, 1917.

Assets

Cash in bank		$3,922.16
Accounts receivable:		
Dues		885.00
Inventories		1,150.98
		$5,958.14

Liabilities.

Advances from members and non-members for bindings, Proceedings, dues, etc.	$138.39
Surplus	5,819.75
	$5,958.14

EXHIBIT B.

Statement of Receipts and Disbursements

For the seven months ended Jan. 31, 1917.

Receipts.

Balance July 1, 1916			$2,015.35
Dues		$4,010.00	
Subscriptions		200.00	
Publications:			
Proceedings	$119.98		
Standards	10.25		
Journals	1.00		
		131.23	
Interest on bank deposit		56.30	
Bindings		14.50	
Miscellaneous		2.00	
Total receipts			4,414.03
			$6,429.38

Disbursements.

Proceedings	$2,332.78	
Membership campaign	102.20	
Miscellaneous expenses	72.24	
Total disbursements		2,507.22
Balance Jan. 31, 1917—Exhibit A		$3,922.16

ABSTRACT OF MINUTES OF MEETINGS OF THE BOARD OF DIRECTION.

Meeting of Board of Direction Held at Auditorium Hotel, Chicago, Ill., February 16, 1916.

Present: Wason, Hatt, Turner, Lesley, Boyer, Anderson, Lindau and Goodell.

Mr. H. D. Hynds was unanimously elected Secretary, to take effect on March 1, 1916.

The Board appointed as Finance Committee, Messrs. Turner, chairman, Wason and Lesley.

The Board appointed on the Executive Committee to serve in addition to the President, Secretary and Treasurer, required by the By-Laws, Messrs. Turner and Boyer.

The Board appointed as the Committee on Publications Messrs. Hatt, Gow, Humphrey, Quimby and Boyer.

The Board authorized the appointment of Committees on Monolithic Concrete Sewers and Aqueducts and on Reinforced-Concrete Standpipes and Water Towers.

The appointment of chairmen of the technical committees was left to the Executive Committee.

Meeting of Executive Committee, Held in the House of the American Society of Civil Engineers, New York City, March 28, 1916.

Present: Wason, Turner, Lesley, Humphrey and Boyer. In the absence of the Secretary, Mr. Boyer was appointed to act as Secretary pro tem.

Minutes of the previous meeting were read and approved.

The Treasurer presented a report showing:

Accounts receivable	$686.50
Cash in bank	798.35
Total	$1,484.85
Accounts payable	$122.18
Contingent assets	1,379.82

MEETING OF EXECUTIVE COMMITTEE, HELD IN THE HOUSE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, JUNE 6, 1916.

Present: Wason, Lesley and Hynds.

The Treasurer presented report showing:

Accounts receivable........................	$140.50
Cash in bank..............................	1,814.85
Total..................................	$1,955.35
No accounts payable.	
Contingent assets.........................	$479.82

MEETING OF EXECUTIVE COMMITTEE, HELD AT 11 BROADWAY, NEW YORK CITY, SEPTEMBER 19, 1916.

Present: Wason, Turner, Boyer and Hynds.

Mr. Clarke, representative of the John C. Winston Co., publishers, was present and reported on Vol. XII of the Institute Proceedings.

On motion, it was decided that there should be a total publication of Vol. XII of 600 copies—450 to be bound in paper, 100 in cloth and 50 in half morocco.

On motion, the papers which were printed previous to the convention and distributed to the membership there, are to be reprinted in Vol. XII. It is estimated that the total expense in connection with this will not exceed $500.

It was announced that the canvass of ballots for Nominating committee resulted in the election of C. M. Chapman, chairman, Arthur N. Johnson, Alfred E. Lindau, Richard L. Humphrey and Ernest Ashton.

Place and time of holding the next convention was discussed. It was decided that three days, comprising six sessions, would be sufficient time for this year's convention, and first choice was for Feb. 12, 13 and 14; second choice Feb. 8, 9 and 10.

Mr. Wason read report stating that the paid members for the fiscal year of 1915 were 335. Resigned members for the fiscal year 1916–17 were 22. Paid dues for the year 1916–17 were 143.

Committee on Reinforced-Concrete Bridges and Culverts, Mr. McCullough, chairman, has requested 150 reprints of the committee's report, and motion was made by Mr. Boyer and seconded by Mr. Turner that these copies should be furnished the chairman. Estimated total cost will not exceed $15.

Discussion was promoted by President Wason regarding proof-reading services of Vol. XII, and whether this should be assumed by the Secretary. It was finally decided that this proof-reading service was not a part of the editorial work and should be charged against the Institute affairs.

Requests on behalf of some concerns for reprints of committee reports were discussed, and it was decided that the Institute would furnish reprints

of their committee reports at cost plus 25 per cent above printer's bill, this 25 per cent to cover overhead of the Institute in connection with this service.

Mr. Lesley suggested that after completion of all volumes in published form, a financial statement of the Institute be sent to financial subscribers of the Institute, to show that the money they had contributed was properly expended. This action was adopted by the Board.

A membership campaign was approved by the Board and it was decided that the members present would each contribute a list of contractors throughout the United States as possible future members of the Institute, and the Secretary was to carry on a subscription campaign by correspondence.

A tentative program for the next convention was discussed, and work is to proceed at once for arranging a complete and interesting program.

MEETING OF EXECUTIVE COMMITTEE, HELD AT 11 BROADWAY, NEW YORK CITY, JANUARY 18, 1917.

Present: Wason, Lesley, Turner, Boyer and Hynds.

President Wason submitted report on the membership campaign covering a period of from July 1, 1916, to January 15, 1917, inclusive: 18 new members were admitted, 13 old memberships renewed and 8 new and renewed library orders.

On motion, it was approved to appropriate another $100 for continuance of the membership campaign, as figures submitted by Mr. Wason showed that the expenses in connection with the campaign were only about 20 per cent of the revenue obtained.

Mr. Wason reported that the net cash on hand, deducting all bills payable and including the cost of the preprints for the next convention, was $3,494.85.

Mr. Turner suggested the appointing of a committee to investigate the Mullen & Buckley warehouse fire at Far Rockaway, L. I., which occurred on November 10, 1916. Mr. Humphrey was appointed chairman and Mr. Wason and Mr. Humphrey were to arrange for the other members of the committee which should be composed of not less than five nor more than seven members.

Mr. Wason read a draft for the annual report of the Board to the Institute, which was approved.

Invitations to hold future conventions at San Francisco, Cal., Richmond, Va., and Asbury Park, N. J., were read. No action was taken.

MEETING OF BOARD OF DIRECTION, HELD AT HOTEL LA SALLE, CHICAGO, ILL., FEBRUARY 8, 1917.

Present: Wason, Humphrey, Hatt, Turner, Boyer, Ashton, Gow, Anderson, Hynds.

On motion, the acts of the Executive Board as recorded in minutes of past year meetings were approved.

President Wason reported that the manuscript comprising Vol. IX (which covers the proceedings of the 1912 convention) was about two-thirds in page proof and that the remainder would be complete in this form during the week. It was further stated that Vol. IX complete and ready to send to the membership would be completed by March 1st.

President Wason reported that manuscript comprising Vol. X (which covers proceedings of the 1914 convention) was completely edited and in printer's hands. Printer promises this volume will be ready to send to membership by April 1st.

On motion by Mr. Humphrey it was decided to refer the decision regarding the publishing of the remaining matter of Vol. XI to the Publication Committee to report to the Executive Committee the character of the papers comprising this unpublished portion of Vol. XI and to obtain estimate covering cost of publishing same; action to be taken by Executive Committee on recommendation of Publication Committee.

Mr. Ashton moved to authorize publication of Vol. XI subject to recommendation of Publication Committee. The motion was recorded and approved.

Mr. Humphrey moved that it hereinafter be considered as the policy of the Institute to publish all papers presented at their conventions with their properly edited discussions. Motion recorded and approved.

A list of 15 applicants was accepted to membership.

Secretary read yearly report of Board of Direction together with Auditor's report and same was approved.

Mr. Boyer moved that the Auditor's Report be presented at convention and same published in 1917 proceedings. Motion approved.

Mr. Humphrey suggested Committee on Far Rockaway Fire have an appropriation to withstand expenses of express, photos, etc. Mr. Boyer moved $100 be appropriated to cover expense of investigation of Far Rockaway fire. Motion approved.

Professor Hatt moved the Far Rockaway Fire Committee communicate to the Executive Board an outline of its work and probable cost of its completion. Motion approved.

MEETING OF BOARD OF DIRECTION, HELD AT HOTEL LA SALLE, CHICAGO, ILL., FEBRUARY 10, 1917.

First meeting of the new Board of Direction after election of officers for the ensuing year at the convention, Chicago, February, 1917.

Present: Anderson, Ashton, Boyer, Hatt, Humphrey, McCullough, Wason, Hynds.

On motion, it was decided to continue the personnel of the Finance Committee for the ensuing year (W. K. Hatt, R. W. Lesley and H. C. Turner).

On motion, Mr. Frank C. Wight was appointed chairman of Publica ion Committee for the ensuing year. The President will apoint the other members of the committee at an early date.

On motion, a list of 14 applicants for membership to the Institute was approved.

On motion, the Secretary was instructed to notify the Girard Trust Co., Philadelphia, Pa., of the personnel of officers of the Institute for the ensuing year.

On motion, the Publication Committee was instructed to prepare pamphlet of the Institute for general distribution covering:

(1) Membership List, (2) List of Institute Standards, (3) Constitution and By-Laws, (4) Aim of the Institute.

Letter was read from Mr. H. B. Alvord stating that he would continue the routine secretarial duties of the Institute (including the keeping of the books) for $10 per week. On motion, the President was authorized to retain Mr. Alvord as a temporary arrangement for the execution of the above duties.

On motion, a special committee of Messrs. Turner (chairman), Humphrey, Wason and Boyer, was appointed to investigate and report regarding permanent headquarters and arrangement for the secretarial duties of the Institute and the possibility of combining the Institute secretarial duties with some other technical association.

On motion, the President was authorized to select and appoint an editor and supervise publication of the Proceedings of the 13th Convention of the Institute and a sum of not to exceed $400 was set aside for this work, same to include all except printers' services in connection with the publishing of the entire proceedings in one volume form.

On motion, it was decided to print no *Journal* this ensuing year.

On motion, the appointing of the personnel of the Technical Committees be referred to the Executive Board.

On motion, all other Institute special committees were continued with the exception of the following: Committee on Publicity, discontinued, Committee on Revetment Work, discontinued.

Be it known that by consensus of opinion of those present at this meeting that the Concrete Institute has contracted no obligation whatsoever to pay any part of the expense for publishing of the report of the Joint Committee on Reinforced Concrete.

On motion, the Publication Committee was instructed to take such action immediately as is necessary to determine whether or not they can avail themselves of the use of the type used in printing the Joint Committee Report for reprinting same in our proceedings. Motion approved.

On motion, permission was granted the technical press to publish in full papers presented at the convention and subsequent thereto, provided proper credit was given the Institute.

On motion, the Secretary was allowed to act on his own judgment regarding method in which the technical press was to get copies of the papers as presented and without any cost to the Institute.

Resignation of chairman of Committee on Insurance was accepted.

ATTENDANCE, THIRTEENTH ANNUAL CONVENTION.

D. A. Abrams, Chicago, Ill.
E. L. Alexander, Indianapolis, Ind.
L. H. Allen, Boston, Mass.
J. H. Anderson, Pittsburgh, Pa.
W. P. Anderson, Cincinnati, O.
W. S. Anderson, Chicago, Ill.
R. C. Angevine, Coldwater, Mich.
Ernest Ashton, Allentown, Pa.
P. J. Asselin, Minneapolis, Minn.
W. M. Atcheson, Syracuse, N. Y.
B. D. Barker, Chicago, Ill.
W. A. Barnhart, Salem, S. D.
A. W. Barrean, Chicago, Ill.
E. W. Barrows, Cleveland, Ohio.
George S. Bartlett, Chicago, Ill.
J. F. Base, Maywood, Ill.
J. P. Beck, Chicago, Ill.
A. B. Becker, Chicgo, Ill.
J. Bentley, Toledo, O.
J. E. Bergquist, Chicago, Ill.
A. J. Biddell, Walkerville, Ontario, Can.
S. J. Binswanger, Chicago, Ill.
S. Birch, Fargo, N. D.
C. L. A. Bockemohle, Chicago, Ill.
George W. Bond, Chicago, Ill.
Ira Bora, Chicago, Ill.
Ed. D. Boyer, New York, N. Y.
E. W. Boynton, Muscatine, Ia.
W. D. Brewer, Pittsburgh, Pa.
F. T. Brown, Chicago, Ill.
H. W. Brown, Cambridge, Mass.
R. P. Brown, Urbana, Ill.
F. J. Cassidy, Chicago, Ill.
L. O. Chamberlain, Chicago, Ill.
C. M. Chapman, New York, N. Y.
A. P. Clark, Buffalo, N. Y.
J. M. Clyne, Maple Park, Ill.
L. R. Cobb, Montclair, N. J.
A. B. Cohen, Hoboken, N. J.
F. E. Colburn, Soperton, Mo.
W. A. Collings, Kansas City, Mo.
T. L. Condron, Chicago, Ill.
J. E. Conzelman, St. Louis, Mo.
J. G. Cooney, Chicago, Ill.
R. W. Crum, Ames, Ia.
H. A. Davis, Washington, D. C.
H. K. Davis, Knoxville, Ia.
H. Dean, Chicago, Ill.
W. W. DeBerard, Chicago, Ill.
F. K. Deinboll, Cleveland, O.
C. E. DeLemo, Chicago, Ill.
E. A. Dolan, Chicago, Ill.
W. J. Driscoll, Appleton, Wis.
H. H. Edwards, Urbana, Ill.
J. G. Ellindt, Rochester, N. Y.
H. G. Garvey, Appleton, Wis.
C. S. Gedney, Chicago, Ill.
W. D. Gerber, Chicago, Ill.
A. J. Gertzke, La Crosse, Wis.
S. J. Gettleson, New York, N. Y.
C. D. Gilbert, Detroit, Mich.
H. F. Gonnerman, Champaign, Ill.
C. R. Gow, Boston, Mass.
S. H. Graf, Corvallis, Ore.
P. J. Greaves, Chicago, Ill.
J. G. Greeman, Chicago, Ill.
J. E. Grimes, Chicago, Ill.
M. Grodsky, Chicago, Ill.
A. Hagener, Chicago, Ill.
J. S. Hanley, Chicago, Ill.
E. S. Hanson, Chicago, Ill.
O. E. Harder, Chicago, Ill.
W. K. Hatt, Lafayette, Ind.
L. C. Hawk, Nazareth, Pa.
L. H. Hawblitz, Toledo, O.
F. C. Hegley, Delaware, O.
O. H. Henschel, Chicago, Ill.
A. G. Higgins, Kansas City, Mo.
L. S. Hillebrand, Toledo, O.
G. E. Hillsman, Chicago, Ill.
A. Holinger, Chicago, Ill.
G. A. Hool, Madison, Wis.
W. W. Horner, St. Louis, Mo.

R. L. Humphrey, Philadelphia, Pa.
W. H. Hurley, Chicago, Ill.
H. D. Hynds, New York, N. Y.
C. E. Ireland, Birmingham, Ala.
A. N. Johnson, Chicago, Ill.
T. H. Johnson, Sioux City, Ia.
P. T. Kalman, Chicago, Ill.
W. M. Kallosch, Elgin, Ill.
H. J. Kamp, Jr., Chicago, Ill.
H. S. Katzenberg, Chicago, Ill.
F. Kellam, Chicago, Ill.
H. D. Kerr, Chicago, Ill.
W. E. King, Chicago, Ill.
W. M. Kinney, Chicago, Ill.
P. Kircher, Chicago, Ill.
R. R. Kitchen, Wheeling, W. Va.
A. H. Krom, Chicago, Ill.
G. W. Krowl, Chicago, Ill.
W. S. Lacher, Chicago, Ill.
Jos. I. Lambie, Pittsburgh, Pa.
H. A. LaRoy, Chicago, Ill.
L. J. Larson, Champaign, Ill.
O. Lartar, La Salle, Ill.
J. W. Lathers, Beloit, Wis.
E. A. Lawrence, Chicago, Ill.
L. H. Lehman, Urbana, Ill.
L. C. Letzkus, Pittsburgh, Pa.
J. H. Libberton, Chicago, Ill.
E. H. Lichtenberg, Milwaukee, Wis.
A. E. Lindau, Buffalo, N. Y.
F. A. Little, Fond du Lac, Wis.
J. Loerhart, Cleveland, O.
N. Loitved, Bemidji, Minn.
R. Loitved, Bemidji, Minn.
A. R. Lord, Chicago, Ill.
M. W. Loving, Chicago, Ill.
J. W. Lowell, Jr., Chicago, Ill.
C. W. Lundoff, Cleveland, O.
H. C. Lynch, Independence, Ia.
J. McCarty, Kaukeuna, Wis.
E. McCullough, Evanston, Ill.
W. A. McIntyre, Youngstown, O.
K. K. MacKenzie, Chicago, Ill.
F. R. McMillan, Minneapolis, Minn.
E. S. Macgowan, Minneapolis, Minn.
R. Mande, Chicago, Ill.
G. B. Mann, Chicago, Ill.
J. B. Marcellus, Kansas City, Mo.
J. Marr, Chicago, Ill.
J. B. Marsh, Des Moines, Ia.
E. N. Mattson, Chicago, Ill.
A. J. Maynard, State Farm, Mass.
J. H. Mayne, Council Bluffs, Ia.
E. J. Mehren, New York, N. Y.
L. J. Mensch, Chicago, Ill.
B. A. Meyer, New York, N. Y.
R. T. Miller, Chicago, Ill.
W. L. Miner, Richland Center, Wis.
L. N. Moeller, Chicago, Ill.
E. J. Moore, New York, N. Y.
F. A. Moorhead, West Jefferson, O.
H. H. Morgan, Chicago, Ill.
G. Moritz, Peoria, Ill.
C. V. Mueller, Chicago, Ill.
G. Nagy, Pittsburgh, Pa.
G. G. Nelson, Elgin, Ill.
J. L. Nelson, Chicago, Ill.
J. B. Nichelson, Toronto, Ontario, Can.
A. H. Ogle, Chicago, Ill.
J. B. Orr, Miami, Fla.
H. G. Overholt, Minneapolis, Minn.
A. F. Owen, Chicago, Ill.
H. S. Owen, Chicago, Ill.
J. E. Paas, Grand Rapids, Mich.
B. S. Pease, Chicago, Ill.
B. Pempsky, Urbana, Ill.
S. Playford, Elgin, Ill.
W. G. Potter, Chicago, Ill.
C. W. Powell, Chicago, Ill.
F. G. Pulley, Chicago, Ill.
A. C. Raymer, St. Paul, Minn.
W. K. Reed, Waynesburg, Pa.
Peter Renn, Kaukeuna, Wis.
O. J. Reynvaan.
J. F. Rhodes, Montreal, Quebec, Can.
D. Richter, Easton, Pa.
D. M. Riff, Chicago, Ill.
C. A. Roberts, Galien, Mich.
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Floyd Rogers, Newton, Ia.
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J. E. D. Sayler, Cleveland, O.
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W. A. Slater, Urbana, Ill.
W. H. Smeaton, Chicago, Ill.
S. Blaine Smith, Chicago, Ill.
C. D. Smith, Chicago, Ill.
E. B. Smith, Washington, D. C.
Edward Smulski, New York, N. Y.
G. A. Somerville, Chicago, Ill.
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George D. Steele, Philadelphia, Pa.
Edward A. Steele, Philadelphia, Pa.
J. G. Stemle, New York, N. Y.
William Stoecker, Maplewood, Ill.
A. G. Stone, Chicago, Ill.
A. E. Surman, Moline, Ill.
W. S. Tail, Lamont, Ill.
H. R. Talbert, Hamilton, O.
A. N. Talbot, Urbana, Ill.
K. H. Talbot, Pittsburgh, Pa.
Fred Tarrant, Chicago, Ill.
E. H. Tashjian, Milwaukee, Wis.
R. L. Templin, Champaign, Ill.
H. V. Tennant, Portage, Wis.
B. A. Thrift, Chicago, Ill.
W. S. Thomson, Buffalo, N. Y.
Sanford E. Thompson, Boston, Mass.
M. C. Tobiss, Chicago, Ill.
A. C. Toner, Pittsburgh, Pa.
E. E. R. Tratman, Chicago, Ill.
W. F. Tubesing, Milwaukee, Wis.
H. C. Turner, New York, N. Y.
J. Ubbint, Port Washington, Wis.
J. J. Ubbint, Port Washington, Wis.
W. Leroy Ulrich, Hartford, Conn.
F. R. Walker, Chicago, Ill.
P. G. West, Milwaukee, Wis.
F. J. Weis, Indianapolis, Ind.
B. Weiss, Omaha, Neb.
B. O. Wheeler, Chicago, Ill.
F. Whipperman, Omaha, Neb.
A. J. Whipple, Chicago, Ill.
H. Whipple, Detroit, Mich.
F. C. Wight, New York, N. Y.
B. Wilk, Chicago, Ill.
M. S. Willens, Chicago, Ill.
F. L. Williamson, Kansas City, Mo.
L. C. Willinan, Chicago, Ill.
J. W. Wilson, St. Charles, Ill.
W. S. Wing, Pittsburgh, Pa.
C. Wingstrom, Chicago, Ill.
M. C. Winoken, Chicago, Ill.
B. E. Winslow, Chicago, Ill.
A. M. Wolf, Melrose Park, Ill.
C. M. Wood, Chicago, Ill.
William Wood, Ada, Mich., R. D. 4.
J. Wurder, Minneapolis, Minn.
C. O. Yeager, Danville, Ill.
H. J. Zove, Youngstown, O.

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APPENDIX.

Through an oversight the discussion of some of the papers presented to the Twelfth Annual Convention of the American Concrete Institute, February 14–17, 1916, were omitted in the Proceedings, Vol. XII. These discussions are given in the following pages.—EDITOR.

Discussion of Paper Entitled "Reinforced Concrete Columns," by Pierce P. Furber, Proceedings, American Concrete Institute, Vol. XII, P. 181.

Mr. Slater. Mr. W. A. Slater.—In Table VI of this paper apparently there has been an error in the computation of the bearing value of a column under the specifications of the Joint Committee. It is seen that in this table under the Joint Committee specification the 24-in. column having four $1\frac{1}{8}$-in. round bars and no spiral has a total bearing value of 324,000 lb. The main error seems to lie in the use of the wrong unit-stresses for the Joint Committee. This column seems to have been calculated with the same unit-stresses for the concrete and for the steel as are allowed for a column having spiral reinforcement in addition to the longitudinal reinforcement.

This column has only 0.9 per cent reinforcement and under a strict interpretation of the Joint Committee report it would not be allowed at all. However using the working stress permitted by the Joint Committee where no spiral is used and with the minimum allowable longitudinal reinforcement (1 per cent) the column would be good for only 225,000 lb. instead of 324,000 lb. as given in the table. This table gives the bearing value under the C. A. P. Turner specification as 192,000 lb. This matter is of some importance as indicating the relative values placed by the Joint Committee and by the Turner specifications upon a column which in some quarters is not in very good repute, the "rodded column" having no spiral.

Again, the column chosen for this comparison can hardly be said to be of representative design. It has less reinforcement than is allowed by thirteen of the twenty-five authorities quoted in Table I which specify a minimum amount of longitudinal reinforcement. If 4 per cent of longitudinal reinforcement has been used in this calculation the position would have been reversed. The Joint Committee specification would allow a bearing value of 317,000 lb. and the Turner specifications would allow 329,000 lb.

Mr Godfrey. Mr. Edward Godfrey.—The writer is thoroughly in accord with Mr. Furber in his condemnation of the rodded column and he hopes that this convention will eliminate this wreck breeder, this criminal element of design, from all of its standards. He is not, however, in accord with Mr. Furber's and Mr. Turner's exalted views as to the safe carrying capacity of hooped columns in the matter of unit stresses to be allowed.

As the writer pointed out in the "Proceedings" of the American Society of Civil Engineers, April, 1914, p. 1092, the average loads on plain and hooped columns at the first sign of failure are almost exactly the same. Prof. Talbot, in Bulletin 20, University of Illinois Experiment Station, p. 29, says: "At a load equal to that which would cause failure in a plain concrete column, or a little above, the concrete over the spacing bars begins to scale and this is soon followed with a scaling and shelling off of the surface of the column over the hoops everywhere."

Mr. S. E. Thompson, in his book, p. 495, says: "The surface of the concrete outside of the hooping will begin to crack at a loading corresponding to the breaking load of an unhooped column." **Mr. Godfrey.**

In view of these plainly proved facts it is impossible to see how there can be any justification for high units on the concrete of a hooped column, or of any compression allowance for the vertical steel, or for any compression allowance for the coil in a column.

Discussion of Paper Entitled, "A Further Discussion of Steel Stresses in Flat-Slab Floors," by Henry T. Eddy, Proceedings, American Concrete Institute, Vol. XII, P. 281.

Mr. Slater.

Mr. W. A. Slater.—In the analysis given in the first part of this paper, Prof. Eddy commits himself to the correctness (for at least one special case) of the equation of applied bending moment given by Mr. John R. Nichols in a paper before the American Society of Civil Engineers.* If Prof. Eddy's and Mr. Nichols' analyses assume point supports both will give $1/8\ Wl$ as the moment at the support in each of the two rectangular directions. This makes it look as though the paper had a basis upon which many could agree. But after having made this analysis Prof. Eddy abandons it and uses reasoning and arrives at conclusions which seem incredible. Among the first of these features is the claim that because two bands of reinforcing steel lie at right angles each to the other, the two bands act as a solid steel plate of the same weight as the weight of these two cross bands within the area of their intersection. This is so much like a fairy tale, that I do not believe that its correctness needs to be denied; I believe that the mere statement of the conclusion is sufficient to indicate its absurdity.

I think also the statement that, of the work of deformation "one-half is expended in deforming the concrete and the other half in deforming the steel" —and that therefore "the steel will be subjected to only one-fourth as great stresses in such a slab as would be calculated in accordance with beam theory, carries with it its own refutation and is unworthy of serious consideration.

After all this to consider that Poisson's ratio may come in to reduce the stress by one-fourth of the remaining so that we have a total of three-sixteenths of the total applied moment to be resisted by the steel brings the matter to a climax of extravagant claims. Mr. C. A. P. Turner† says of the plate of reinforced concrete that "its coefficients of necessity are dependent on the connecting link bonding the heterogeneous materials together." I believe Prof. Eddy, like Mr. Turner, must be talking about "co-efficients of necessity" instead of co-efficients of bending moment.

Having made these observations about the analysis, I must take some exception to the manner in which he has applied the results of tests to prove the correctness of his analysis. After he had taken as a free body half a slab between columns, cutting out around the edge of the capital, he has assumed that the two diagonal bands at the middle of the panel have the same effect in resisting moment as though there were just one band and, as though it were parallel to the span. This is one place where it is not necessary to make any assumption. The steel stress in the diagonal band will have a component in the direction of the span proportional to the cosine of the angle between the direction of the span and the direction of the bars. On this basis the

* Trans. A.S.C.E., Vol. LXXVII (1914), p. 1670.

† Trans. A.S.C.E., Vol. LXXVII, p. 1421.

resisting moment of the steel stresses for the Deere and Webber Building is 23 per cent greater than that given by Prof. Eddy. Similar corrections should be made in the other tests quoted and all the corrections would reduce the excess of the applied bending moments over the resisting moment of the steel, below the amount arrived at by Prof. Eddy. **Mr. Slater.**

There is another feature to which attention should be called, because I think it is likely to be overlooked by many; a proper moment coefficient for design would not be obtained by summing up the resisting moments of all the stresses on the sections at the support and mid-span and placing the total resisting moment equal to the applied bending moment, because we design for a uniform stress in all the bars at all sections of maximum moment, whereas the coefficient is obtained by using the sum of all the moments due to greatly varying stresses which were found in the tests. Since the stresses were not uniform in the test on which the conclusions are based the greatest stress found in a floor designed by such a coefficient should be as much higher than the average stress as the maximum stresses in the test were higher than the average stress in the test. If floors were built similar to those described in this paper except that they used only enough steel to resist the total moment found by the tests of these buildings the maximum steel stresses to be expected for the new design under working load would be as given in the accompanying table.

Building.	Observed Stress, lb. per sq. in.		Probable Stress in New Design, lb. per sq. in.
	(1)	(2)	(3)
Deere and Webber	18 900	11 900	26 500
Franks	5 360	3 585	24 000
Larkin	24 200	13 800	28 100
Northwestern Glass	14 600	11 400	20 500
St. Paul Bread Co	17 200	11 500	23 700

Column (1) gives average for entire band at section of highest stresses.

Column (2) gives average for all bands.

Column (3) gives probable stress for entire band at section of highest stresses if slab were designed for only the total moments of resistance of steel stresses found in tests. Letting f_1 = stress given in column (1); f_2 = stress given in column (2); f_3 = stress given in column (3) and 16000 = working stress in new design.

$$f_3 = \frac{f_1}{f_2} 16000$$

The ratios of these maximum stresses to the design stress are the same as the ratios of the maximum stresses to the average stress as found in the tests of the buildings.

Mr. Slater. In Prof. Eddy's Flat Plate Theory* he presented an equation for the stress at the edge of the capital

$$(43)\quad f_s=\frac{WL_1(L_1+L_2)}{800d_3A_1L_2}\left(3\frac{B_1^2}{L_1^2}-1\right)$$

Apparently in comparing his test of the Northwestern Glass Company Building with the theory it was found that the stresses in the bars where they cross the edge of the capital were not according to equation (43) but were about three times as large and it was necessary to find another equation which would represent those stresses. He used as the basis of this new equation the assumption (an assumption which in itself may be justified) that the stress at A at the edge of the capital is equal to the stress at C across the edge of the panel, but out at the side of the belt. On this basis he arrives at equation (a)

$$f_s=\frac{WL_1(L_1+L_2)}{800d_3A_1L_2}\left(3\frac{L_1^2}{B_1^2}-1\right)$$

The mathematical gymnastics necessary to derive equation (a) from (43) are nothing short of ludicrous (Prof. Eddy has not shown the derivation), but even after obtaining equation (a) the stresses used for comparative purposes were those computed by this formula for the center of the column instead of those across the edge of the capital. If in the results of the International Hall Test the computed stress at the edge of the capital had been compared with the observed stress at the same place the observed stress would still have been 75 per cent greater than the computed stress. If equation (43) had been used for computing the stress the excess of the observed over the computed stress would have been 250 per cent. Yet equation (43) still stands in Prof. Eddy's more recent analysis† which has been published since the analysis here referred to was given.‡

Further, in making use of the test data of International Hall stresses observed at the design load were used for comparison with computed stresses when stresses for a load of two and one-half times the design load were available for this purpose. Also a position on the slab was selected for this comparison where the stresses were much less than the representative stresses in another part of the slab. If the most significant comparison had been made it would have been found that the observed stress was from five to ten times as great as the stress computed by Prof. Eddy's formula applicable to that place.

The above discussion has shown the misuse of even the author's own equations in interpreting his test data. It is of importance also to call attention to the character of the equations themselves.

In equation (a)

$$f_s=\frac{WL_1(L_1+L_2)}{800d_2A_1L_2}\left(\frac{3L_1^2}{B_1^2}-1\right)$$

* "Flat Plate Theory of Reinforced Concrete Flat Slabs," Eq. 43, p. 33.

† Concrete Steel Construction, Part 1, p. 190.

‡ Trans. A.S.C.E., Vol. LXXVII, p. 1434.

Mr. Slater.

In this equation $B_1 = 2x$ in which x is the distance from the center line of the panel to the point for which the stress is computed. Therefore, the more general form of the equation is

$$f_s = \frac{WL_1(L_1+L_2)}{800d_2A_1L_2}\left(\frac{3L_1^2}{4x^2}-1\right)$$

and equation (a) is the special case in which $x = \frac{B}{2}$. This indicates that the stress is independent of the value of y and makes it clear that logically equation (a) gives the stress in the reinforcement where it crosses the edge of the capital. As the capital becomes larger B becomes smaller and according to this equation the stress across the edge of the capital becomes larger. By this formula the stress in the reinforcement across the edge of the capital is 18 per cent greater where the capital is 0.25 of the span than when it is only 0.2 of the span, and to obtain the smallest stresses there the columns should terminate as pin points instead of enlarged heads. No complex analysis can justify such a result. It is interesting that when the column diameter is equal to the span the stress over the edge of the column becomes infinite.

In view of the extravagant nature of the claims and assumptions on which Prof. Eddy bases his analysis, the faulty uses which he has made of test data in a previous publication for comparison with the analysis, and the absurd character of the equations at which he arrives as a result of such analysis it does not seem that this paper merits serious consideration.

Prof. Talbot.

PROF. A. N. TALBOT.—Attention should be called to the method used by Dr. Eddy in obtaining the low moment coefficients advocated by him for the flat slab. After deriving by mechanics the expression $\frac{1}{8}WL$ as the value of the moment which must be resisted by the sum of the positive moment on a center section of the panel and the negative moment on an end section of a panel, he cuts the coefficients down to one-fourth of their real value by two simple expedients. The first is (third paragraph, page 286) that if two layers of rods or bars cross each other at right angles and are embedded in a matrix of concrete the effect is the same as if there were a solid sheet of metal having the same amount of material as exists in the two sets of rods. In other words, a reinforcing rod is as effective sidewise as it is in the direction of its length. This multiplies its usefulness by two, or, as he uses it, divides the required moment by two—a conclusion which is not borne out by either theory or experiment. The second is (fifth paragraph, page 286) that when a reinforcing rod is embedded in concrete and does not slip in the concrete but stretches under stress, the concrete must stretch as much as the steel, and that, therefore, one-half of the moment must be taken by the concrete in tension (a novel way of considering the effect of the tensile resistance of the concrete), thus reducing the requirement of moment again by two. By this magic he cuts the moment coefficient to one-fourth the value derived in the first part of the paper. The method used is so simple and the reasoning so illogical that it seems unnecessary to do more than call attention to the statements of these paragraphs.

Mr. Godfrey. MR. EDWARD GODFREY.—In this paper Mr. Eddy appears to acknowledge that beam theory has some remote connection with this darling of the reinforced concrete gods, the flat-slab. Ignoring for the time his facility for passing his hand over a bending moment and then showing the audience that it is only half as great as before, he seems to acknowledge that there is such thing as a static limitation to bending moments in a flat-slab. The writer pointed this out in *Engineering News*, Feb. 29, 1912, and at various times since that John R. Nichols, in Trans. Am. Soc. C. E., Vol. 87 (1914), p. 1670, demonstrates the same thing in somewhat different manner. In the first reference cited the writer proposed as a criterion that a flat-slab be considered as supported on two rows of columns and given the test of a slab supported on two lines of girders, than which the flat slab is no better. I have put this proposition definitely to Mr. Eddy twice, and he has sidestepped it twice. Once (Trans. Am. Soc. C. E., Vol. LXXVII, p. 1714) he took care to specify oblong panels with the weak direction or long spans in line of the rows of columns, so that a failure would take place in this direction before the section under consideration was taxed. The other time (Trans. Am. Soc. C. E., Vol. LXXVII, p. 1424) he made the "row" of panels three in number and the end supports walls, which walls, he said, save the mushroom slab from a condition of critical stress.

Another point of note about this paper differing from his former writings is that Mr. Eddy omits to designate as the "true bending moment" that which commercial designers are willing to introduce steel to resist and the "apparent bending moment" as that which actually exists in the slab reinforced.

Mr. Eddy says: "Now the fact is that when belts of rods which lie one across the other are embedded in a bulky concrete matrix, the matrix so ties the crossed rods together as to replace the connections of a continuous sheet and causes the steel to act in conjunction with the concrete like a continuous sheet of steel of the same total weight as the crossed belts. This effect is produced by the bond-shear called into play to prevent the concrete from sliding along the surface of the steel rods."

There is not a single fact in this quotation. It is pure fancy, though it is stated as though it were very generally accepted or at least had been proved. The only way that steel rods could be pulled crosswise is by adhesion of the concrete, and such adhesion to an intensity approaching that allowed on steel is utterly impossible.

Mr. Eddy waves aside the one absolutely simple and satisfying and complete explanation of the results of static tests on flat-slabs, namely, the aid supplied by tension in the concrete, by saying: "But such direct tensile stresses in concrete are not regarded as admissible by any one, because they are not to be relied upon permanently." The reason he gives for thus waving aside a big fact is the very reason why, for safety, it should be considered. It is because tensile stresses are not to be relied upon permanently that they should be given every consideration in the investigation of a flat-slab. To say that tensile stresses cannot be relied upon does not eliminate such stresses in the least degree. They exist and they have a tremendous

Mr. Godfrey.

effect on the strength of a slab under static test. Tension in the concrete of a whole slab does many times more toward supporting a test load than tension in the steel reinforcement. But if the slab be cracked this whole work may be thrown on the steel with serious results.

Mr. Eddy also ignores tension in the concrete in the work performed in deflecting a slab. To attribute all of this work and the resisting of bending moments to steel, when in fact the steel does only a small fraction of it, is calculated to deceive the ignorant into the belief that only a fraction of the steel actually required to take the tensile stresses in a flat-slab need be supplied.

Tests could not be more falsely interpreted than such interpretation as ignores the tensile stresses in the concrete, and standards could not be more unjustly devised than those that give one class of systems so great advantage over another as to allow one to use the full value of tension in the concrete and the other to use none of it.

If six men each drank a glass of whiskey, and five of them were drunkards, it would be just as logical for me to say that the one man drank all six glasses because the other five "are not to be relied upon permanently," as to say, that steel that does one-sixth of a piece of work is doing the whole job because its helpers (concrete in tension) are not to be relied upon permanently.

There is no mystery whatever about the tensile strength of concrete and the assistance that it affords in slabs and beams to resist bending moments. It is exhibited in beam tests, as pointed out by Mr. L. J. Mensch (Trans. Am. Soc. C. E., Vol. LXXVII, p. 1403) in discussing beam tests. He cites one example where, with two-thirds of the ultimate load on a beam the micrometer reading showed only 14,160 lb. per sq. in. on the steel. One with three-quarters of the ultimate test load showed only 19,000 lb. per sq. in. This condition was general. Suppose beam designers should say that this result is caused by "bond-shear" and Poisson's ratio. They would have exactly the same right to do so that the flat-slab proponent has to say it in regard to a flat slab.

As I showed in my paper read here a year ago, in an ordinary case of reinforced slab the tension in the concrete may be actually six times as much as that in the steel. And this tension is resisting bending moments with nearly as great effectiveness per unit as the steel. This fully explains Mr. Eddy's comparison between tests and calculated bending moments where such ratios as 5 to 7 are held out as apparently discrediting the beam theory.

The strength of a whole flat-slab under static load is not mysterious in the least degree, and no new property of steel or concrete need be invented to account for it. But let enough cracks occur in the slab, as has already happened in one very large building of mushroom type, and the steel will get its full share of the tension. This building cracked around the column heads and the slabs sagged several inches.

Mr. Eddy failed to mention, among tests cited those on the Northwestern Glass Co. Building. Mr. Sanford E. Thompson points out (in Trans. Am. Soc. C. E., Vol. LXXVII, p. 1396) that stress in rods across column caps in this mushroom building reached as high as 22,000 lb. per sq. in. for live-load stress only when less than the "safe" load was on the floor.

Mr. Godfrey.

There is a fact in regard to micrometer measurements on steel rods in concrete that investigators have overlooked. Two holes are dug into the concrete until the steel is exposed. Measurements are taken from these two points. These measurements of course tell only the live-load stress, but it is a fact that they may not be measuring the true stress (or increase of stress) in the gaged length. The average increase of stress in the gaged length is the thing that is actually measured; but if there is a crack in the concrete or slippage in a short portion of the rod, the stress here will be very much in excess of the average, possibly several times the average. These facts should be carefully considered in interpreting tests, especially when systems based on those tests are designed in defiance of the laws of mechanics.

Mr. Westergaard.

H. M. WESTERGAARD.—The evidence which I have to give supports those who disagree with Dr. Eddy. It has been procured with equipment of the same kind as was at Dr. Eddy's disposal.

Allow me to quote a few passages from Dr. Eddy's paper, thus summing up what seems to be the quintessence of his results:

> "Now the fact is that when belts of rods which lie across one another are imbedded in a bulky concrete matrix, the matrix so ties the crossed rods together as to replace the connections of a continuous sheet and causes the steel to act in conjunction with the concrete like a continuous sheet of steel of the same total weight as the crossed belts."

After a remark that half of the deflection work is expended in deforming the concrete, it follows:

> "It appears, therefore, that the steel will be subjected to only one-fourth as great stress in such a slab as would be calculated in accordance with beam theory, in which the section of the steel was computed to resist the entire bending movement."

Now Poisson's ratio is introduced, and then follows:

> "So far as tests go, the changes of deformation due to this cause appear to reduce the steel stresses in slabs to three-fourths of what they would otherwise be. . . . It will be observed that, according to this amended theory, the applied bending moments amount to at least $4\times\frac{4}{3}=5\frac{1}{3}$ times those actually to be found on this theory from the elongations of the steel."

The closing remark of the paper implies that theory and tests show such results.

If Dr. Eddy's conclusions are incorrect, it is desirable to have this brought out conclusively, so that their appearance at engineering conventions will not continue. In that case, it will be necessary in this discussion to show that the experiments referred to do not prove the statements quoted, to show that experiments give other results, and to show that theory does not lead to the results presented by Dr. Eddy. I shall confine myself to the last two points.

The experimental evidence which I can present touches incidentally the very point under discussion. The test referred to is still in progress at

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the University of Illinois Engineering Experiment Station, but the data already available give results which definitely disprove Dr. Eddy's claims. The method of testing is as follows: With a specially designed testing machine a uniformly distributed bending moment was applied all around the edge of a slab 4 ft. 8 in. square, thus producing uniformly distributed bending moments all over the slab. The apparatus allows the application of one load along the east and west edges, another load north and south. In that way it was made possible to study the difference of the elastic behavior of the slab under one-axial and biaxial loading. To obtain further comparison, beams were tested along with the slabs.

The slabs are 6 in. thick. The reinforcement consists of two layers of ½-in. bars, across one another and parallel to the sides of the square. The spacing is 3½ in. Four slabs and five beams have been tested to date.* The results obtained may be summarized in the following statements:

The biaxially-reinforced slabs cracked in the tension face with a number of cracks parallel to the edges of the slab and all through the width of the slab. Their nature seems to be exactly the same as that of the tension cracks in the beam. At the yield point one or two of the cracks opened up widely. Assume the case of an increase of the bending load applied along the north and south edges while the load-bending moment in the east and west direction is kept constant. Denote by "longitudinal deformation" the corresponding increase of the deflection in the north-south central plane, and by lateral deformation the corresponding decrease of the deflection in the east-west plane. By definition "Poisson's ratio" for this case of bending is the ratio of this lateral to the longitudinal deformation. Then the results observed were as follows: With low load Poisson's ratio was in most cases about 0.1. With higher loads the ratio was in some cases zero, in some cases a negative Poisson's ratio effect took place, this meaning that an increase of the load north-south caused an increase of the deflections east-west. (The physical explanation of this is possibly that the increase of the north-south load loosened up the concrete along the east-west bars, thus decreasing the bond and thereby also decreasing the concrete tension still remaining between the cracks.)

Further, it was found that the yield-point load and maximum load of the biaxially-bent slab are essentially the same as those of the beam with the same percentage of steel area in the cross-section.

These are the main results of the tests. As a comment on the statement about the appearance of tension cracks extending all through the width of the slab, reference might be made to the fact that these cracks are made out of atmospheric air, hence no more tension or bond shear can be transferred through them than corresponds with the tensile or bond-shear resistance of the atmospheric air under ordinary circumstances. This is to be compared with Dr. Eddy's various statements about the co-operation between the concrete and the steel.

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* Note in June, 1917: The tests were later completed. The results are expected to be published as soon as a few additional data are worked up.

Mr. Westergaard. Dr. Eddy finds the deformations reduced in the ratio of $\frac{3}{4}$ owing to the Poisson's ratio effect. According to the previously published analysis (see Eddy and Turner, Concrete Steel Construction, Minneapolis, 1914), a Poisson's ratio of K gives $1-K_2$ as the reduction ratio of the deflections of flat slabs. $K=0.1$ as found experimentally at the University of Illinois, gives $1-K_2=0.99$, that is, a reduction of only 1 per cent. This is to be compared with Dr. Eddy's 25 per cent.

The result that the carrying power of the biaxially-reinforced slab is about the same as that of the beam, is to be compared with Dr. Eddy's statement that it is $5\frac{1}{3}$ times that much.

Still one point might be made about the Poisson's ratio effect. A positive Poisson's ratio would decrease the deformations of a flat slab, but this does not necessarily imply any increased strength of the structure. A. Föppl tested cubes under biaxial pressure.* Compared with the ordinary compression test there was a decrease of the compressive deformations, this owing to the Poisson's ratio effect, but the tests did not indicate any increase of the ultimate load.

Now let us consider the theory.

Mathematical investigations concerning the flexure of plates have been made since 1815, when a prize was adjudged to Mlle. Sophie Germain. Her paper, "Recherches sur la théorie des surfaces élastiques," was published in Paris, 1821. Three investigations of comparatively recent date should be mentioned:

J. Hadamard: Sur le problème d'analyse relatif a l'équilibre des plaques élastiques encastrées, Memoires des Savants étrangers, Vol. 33, 1907. (Awarded the prize of the French Academy.)

W. Ritz: Uber eine neue Methode, etc., Journal für die reine und angewandte Mathematik, Vol. 135, 1909.

Arpád Nádai: Die Formänderungen und die Spannungen von rechteckigen Platten, Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, Vol. 171–172, Berlin, 1915.

See also A. E. H. Love, Mathematical Theory of Elasticity. Ed. 1906, in particular pp. 468–469. I am making these quotations in order to bring out that the theory of slab flexure is at present in a highly developed stage.

Dr. Eddy's mathematical theory is given in Concrete Steel Construction, by Eddy and Turner, Minneapolis, 1914. At last year's Convention of the American Concrete Institute the writer had the opportunity of offering criticisms of that theory. This was in connection with the discussion of the paper presented by Mr. C. A. P. Turner. Assuming that this discussion will be published in the *Journal* of the American Concrete Institute, it will not be necessary to repeat any of the points.

Tests disprove Dr. Eddy's results. If it is theory that leads to a slab strength which is the four or five double beam strength then I shall offer only one comment to such a theory.

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*Föppl: *Technische Mechanik*, Vol. V, Ed. 1907, pp. 21-22.

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There are three methods of making a theory, and there are also three methods of working out a treatise about the subject of camels. One kind of writers would fit out a complete hunting expedition, and in the land of the wild camels make a study of their lives and habits. Another method is to read up all that has hitherto been written about camels, work that up, and write the book. The third method is, that the writer goes into his room, locks the door, and out of the depth of the inside of his mind he thinks out what camels are. He may find out that a camel has five legs.

A camel was made out of reinforced concrete. Prof. Hatt told its genesis before this convention. The theory of the five double beam strength in slabs is the fifth leg of the reinforced-concrete camel.